SEPTEMBER 1956

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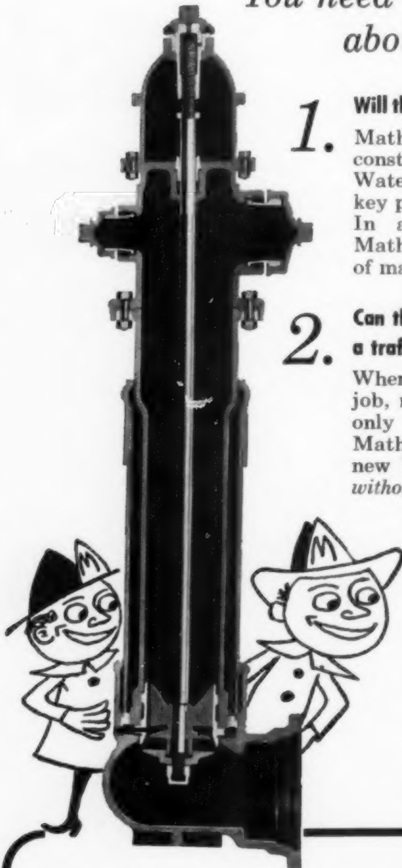
AMERICAN WATER WORKS ASSOCIATION
1881 1956

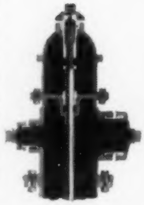
VOL. 48 • NO. 9

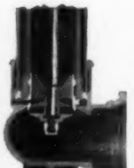
Journal

AMERICAN WATER WORKS ASSOCIATION

In this issue







Journal

AMERICAN WATER WORKS ASSOCIATION

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September 1956

Vol. 48 • No. 9

Contents

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Sep. 12–14—Michigan Section, Burdick Hotel, Kalamazoo. Secretary, T. L. Vander Velde, Chief, Water Supply Section, Michigan Dept. of Health, Lansing 4.

Sep. 12–14—North Central Section, at Hotel Lowry, St. Paul, Minn. Secretary, Leonard N. Thompson, Gen. Mgr., Water Dept., St. Paul 2, Minn.

Sep. 17–19—Kentucky-Tennessee Section, at Hotel Patten, Chattanooga, Tenn. Secretary, J. W. Finney Jr., 553 S. Limestone St., Lexington, Ky.

Sep. 19–21—Ohio Section, at Commodore Perry Hotel, Toledo. Secretary, M. E. Druley, Dayton Power & Light Co., Wilmington.

Sep. 26–28—Wisconsin Section, at Stoddard Hotel, La Crosse. Secretary, L. A. Smith, Supt., Water & Sewerage, City Hall, Madison 3.

Sep. 30–Oct. 2—Missouri Section, at Hotel Governor, Jefferson City. Secretary, W. A. Kramer, Rm. 3, 6th Floor, State Office Bldg., Jefferson City.

Oct. 14–17—Southwest Section, at Marion Hotel, Little Rock, Ark. Secretary, Leslie A. Jackson, Mgr.-Engr., Municipal Water Works, Robinson Memorial Auditorium, Little Rock, Ark.

Oct. 15–16—Canadian Section, Maritime Branch, at Admiral Beatty Hotel, St. John, N.B. Secretary, J. D. Kline, Asst. Mgr. & Chief Engr., Public Service Commission, 62 Lady Hammond Rd., Halifax, N.S.

Oct. 18–20—New Jersey Section, at Hotel Madison, Atlantic City. Secretary, C. B. Tygert, Wallace & Tiernan Inc., Box 178, Newark 1.

Oct. 21–24—Alabama-Mississippi Section, at Battle House, Mobile, Ala. Secretary, Irving E. Anderson, Dist. Engr., Surface Water Branch, USGS, Box 2052, Jackson, Miss.

Oct. 23–26—California Section, at U. S. Grant Hotel, San Diego. Secretary, Henry J. Ongerth, Sr. San. Engr., Bureau of San. Eng., 905 Contra Costa Ave., Berkeley 7.

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Coming Meetings

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Oct. 24–26—Iowa Section, at Hotel Fort Des Moines, Des Moines. Secretary, J. J. Hail, Supt., Water Dept., City Hall, Dubuque.

Oct. 24–26—Chesapeake Section, at Sheraton-Belvedere Hotel, Baltimore, Md. Secretary, Carl J. Lauter, 6955—33rd St., N.W., Washington 15, D.C.

Oct. 31–Nov. 2—West Virginia Section, at Hotel West Virginian, Bluefield. Secretary, Hugh W. Hetzer, Engr. Gen. Office, West Virginia Water Service Co., 179 Summers St., Charleston 1.

Nov. 7–9—Virginia Section, at Chamberlin Hotel, Old Point Comfort. Secretary, J. P. Kavanagh, Dist. Mgr., Wallace & Tiernan Inc., 213 Carlton Terrace Bldg., Roanoke.

Nov. 11–14—Florida Section, at Daytona Plaza Hotel, Daytona Beach. Secretary, Jay D. Roth, City Hall, Miami Beach 39.

Nov. 12–14—North Carolina Section, at Hotel Charlotte, Charlotte. Secretary, Wilbur E. Long Jr., 1615 Bickett Blvd., Raleigh.

Nov. 26–28—Rocky Mountain Section, at Broadmoor Hotel, Colorado Springs, Colo. Secretary, Jack W. Davis, Dist. Mgr., Transite Pipe Div., Johns Manville Sales, Inc., Denver, Colo.

Nov. 29–Dec. 1—Cuban Section, at Cuban Society Engineers Bldg., Havana. Secretary, Laurence H. Daniel, Pres., Laurence H. Daniel, Inc., Baratillo 9, Havana.

OTHER ORGANIZATIONS

Sep. 17–21—Annual International Instrument-Automation Conference & Exhibit, New York, N.Y. For information, write Fred J. Tabery, 250 W. 57th St., New York.

Oct. 8–11—Federation of Sewage & Industrial Wastes Assns., Statler Hotel, Los Angeles, Calif.

Oct. 8–12—National Metal Exposition & Congress, sponsored by American Society for Metals, at Public Auditorium, Cleveland, Ohio.

Oct. 14–17—National Institute of Governmental Purchasing, at Conrad Hilton Hotel, Chicago. A. H. Hall, Exec. Vice-Pres., 1001 Connecticut Ave., N.W., Washington 6, D.C.

Oct. 15–17—Annual Convention, American Gas Assn., Atlantic City, N.J. For reservations, write American Gas Assn. Housing Bureau, 16 Central Pier, Atlantic City, N.J.

Oct. 15–19—American Society of Civil Engineers, Pittsburgh, Pa.

Nov. 12–16—American Public Health Assn., Atlantic City, N.J.

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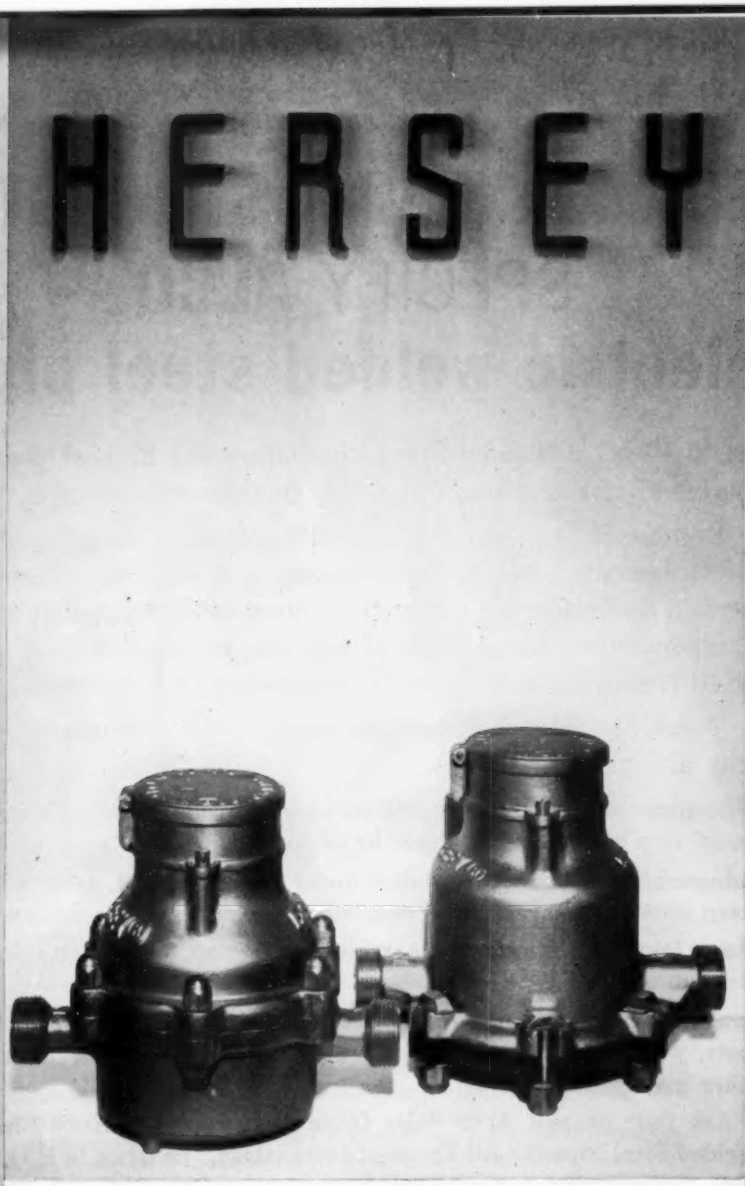
1957

Feb. 3–5—Annual Midwinter Conference, Public Utility Buyers' Group, National Assn. of Purchasing Agents, at Brown Hotel, Louisville, Ky. Chairman, L. G. Wiseley, Michigan Consolidated Gas Co., 415 Clifford St., Detroit 26, Mich.

Mar. 25–29—Western Metals Congress & Exhibition, at Ambassador Hotel and Pan-Pacific Auditorium, Los Angeles, Calif. Managing Director, W. H. Eisenman, 7301 Euclid Ave., Cleveland 3, Ohio.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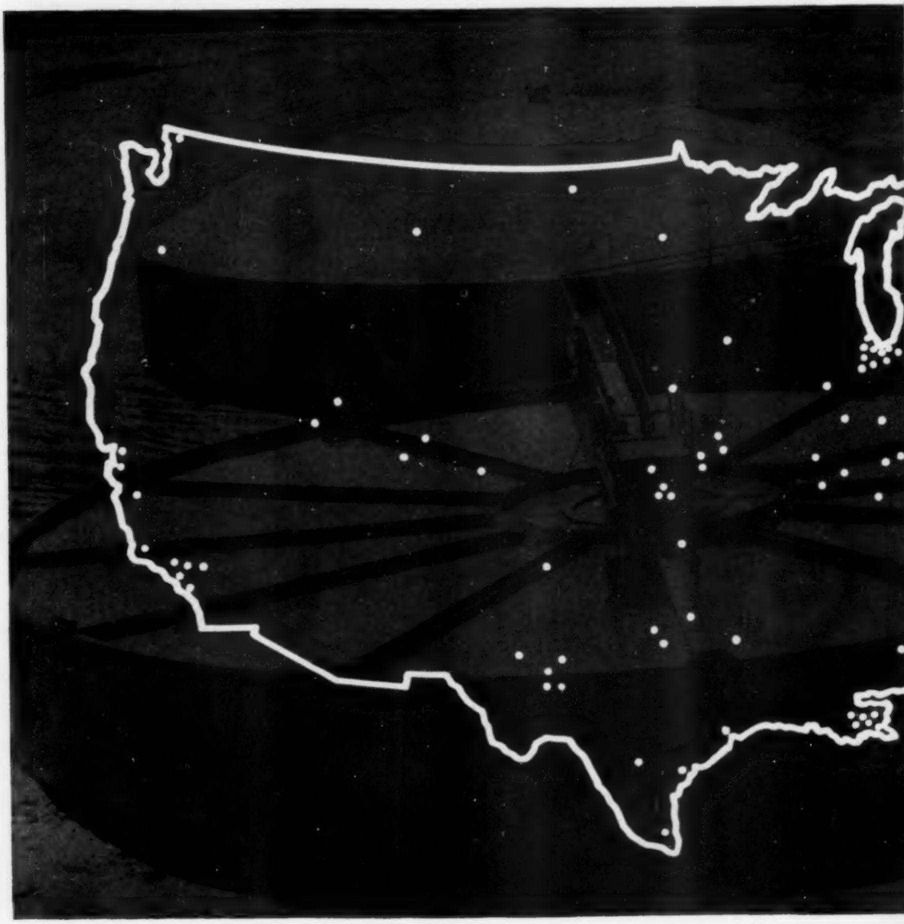

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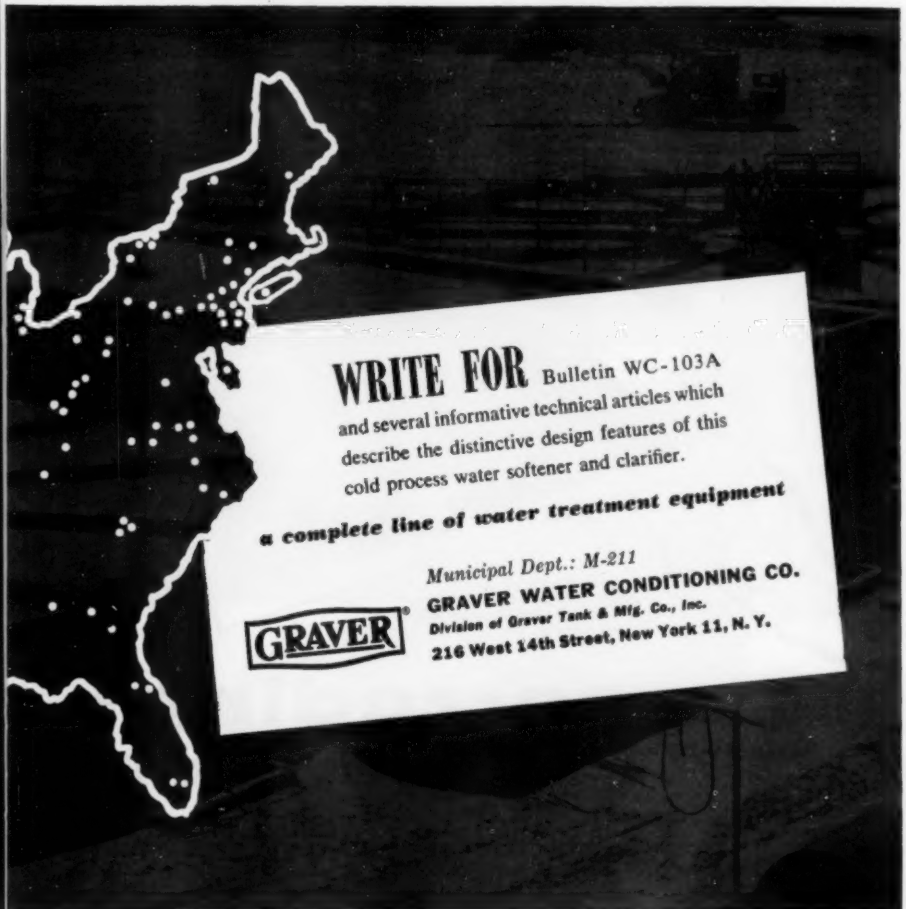

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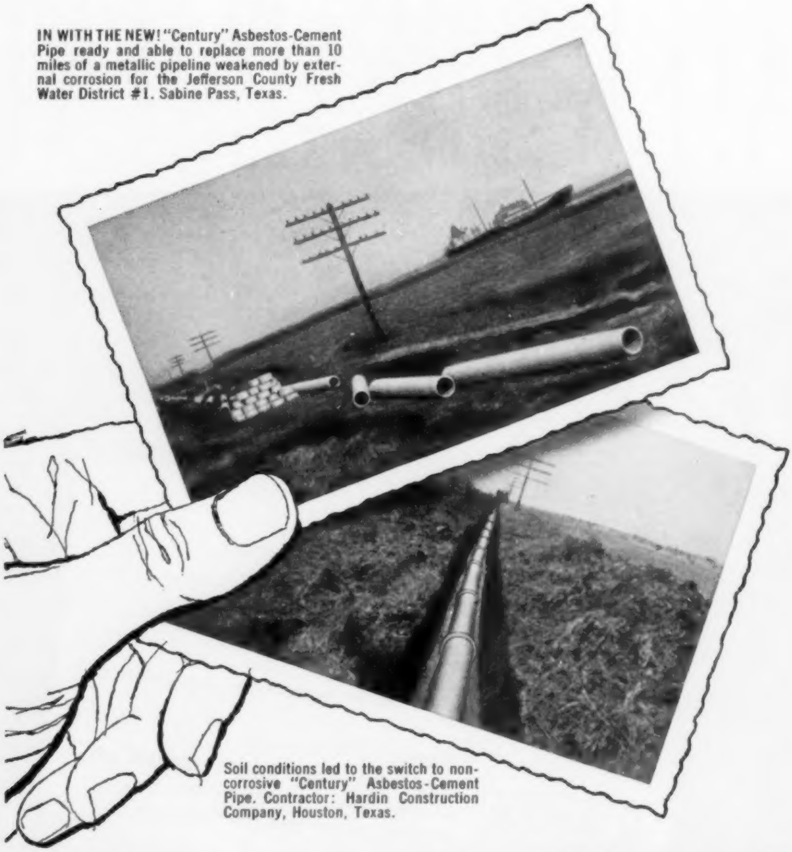

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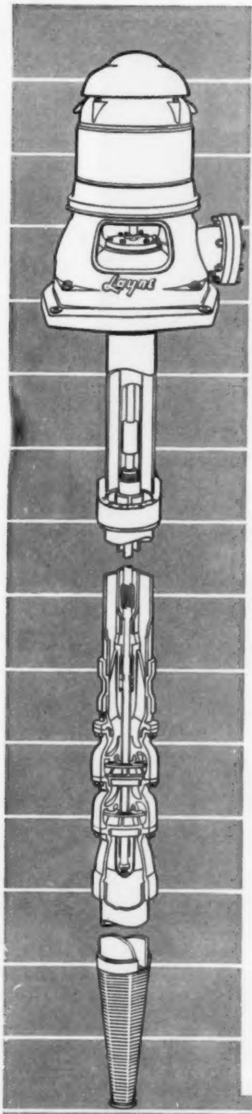

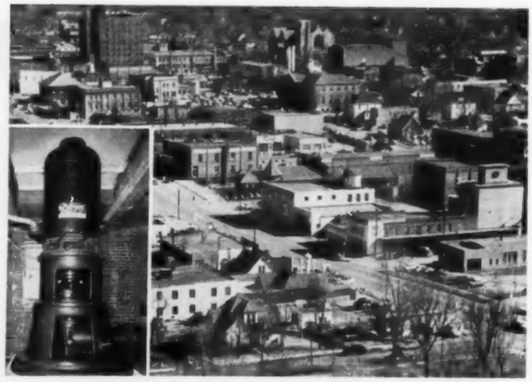


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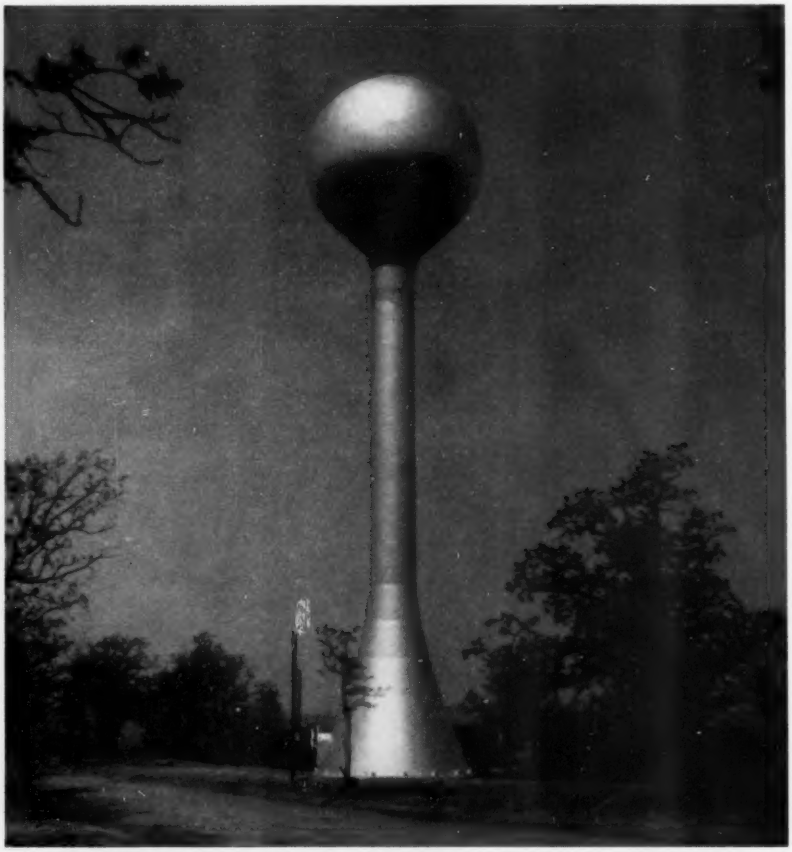

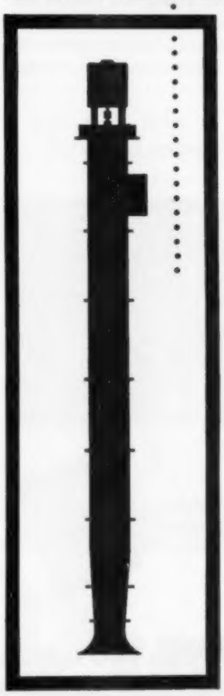

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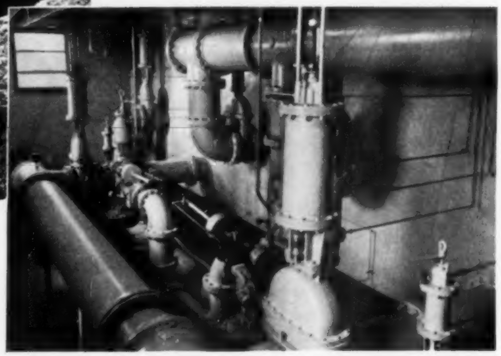

Interior, filter plant pipe gallery: RAMUC Utility Enamel safeguards piping, walls and ceilings.

Journal

AMERICAN WATER WORKS ASSOCIATION

VOL. 48 • SEPTEMBER 1956 • NO. 9

Water Supply Development in the St. Louis Area

Panel Discussion

A panel discussion presented on May 7, 1956, at the Diamond Jubilee Conference, St. Louis, Mo.

The Area's Water Resources—*Charles M. Roos*

A paper presented by Charles M. Roos, Cons. Engr., Belleville, Ill., formerly Mgr., East St. Louis & Interurban Water Co., East St. Louis, Ill.

THE St. Louis Metropolitan Area is regarded as including St. Louis, the three adjoining counties in Missouri, and three counties on the opposite side of the Mississippi River in Illinois (*see* Fig. 1). This comprises a total population of nearly 2,000,000. St. Louis and St. Louis County are separate political entities. This metropolitan area is that in which the Bi-State Development Agency, by special acts of the Missouri and Illinois legislatures and by Congress, has been authorized to function. The organization of this agency was patterned after that of the Port Authority of New York.

Water Supply

The Mississippi, Missouri, Illinois, and Meramec rivers provide the area with an almost inexhaustible supply of water of excellent quality for domestic and industrial uses. The average rate of flow of these rivers for the past 18 years, computed on a monthly basis, has been more than 100 bgd. The ten public supply intake pumping stations within the area, all located on the rivers, provide water for a daily demand of approximately 340 mil gal.

In addition to the unfailing surface supply from the rivers, ground water of year-around low temperatures is available, suitable for cooling purposes and certain other industrial uses. Four ground water producing sections are located within the area, with wells approximately 100 ft in depth; they are the American Bottoms, the Missouri Valley, the Meramec Valley, and the Alton Lake sections, comprising a total of about 400 sq miles. A total of ap-

proximately 130 mgd is now being pumped from the wells, all but about 7 mil gal of which is for industrial uses. Most of this ground water is being taken from the American Bottoms, located just east of the Mississippi River.

The Upper Mississippi River system drains about 23 per cent of the land area of the United States. If Marquette had not made his error, calling the Missouri River a tributary of the main stream rather than the actual upper Mississippi River, the Father of Waters would be the longest river in the world. The combined length of the Mississippi and Missouri rivers is 5,450 miles, of which 4,461 are navigable.

Geologic Glance

During the long geologic past, the Mississippi River scored a channel southwardly through what is now the St. Louis Metropolitan Area. Cutting through Pennsylvanian and Mississippian limestone and shale, the channel reached an average depth of more than 150 ft, with a maximum width of 11 miles. The river now occupies a channel of less than 1 mile in width, and the remainder of the huge basin is filled to a depth of about 100 ft with water-bearing alluvial sediment.

From east to west, there is a consistent upward slope of the non-water bearing underlying consolidated rock stratum, which goes to a great depth. The slope of this stratum is caused by the Ozark Uplift and explains why ground water in the area above the St. Peter Sandstone, except in very limited quantities, can be obtained only in the alluvial deposits in the river valleys. In the St. Louis area, water from the St. Peter sandstone is too highly mineralized for use and is commonly referred to as salt water.

East St. Louis, Ill.—*Stephen C. Casteel*

A paper presented by Stephen C. Casteel, Mgr., East St. Louis & Interurban Water Co., East St. Louis, Ill.

Each day when the river gage at St. Louis is at zero, sufficient water will flow under the historically famous Eads Bridge to supply each of the 160,000,000 people in the United States with 210 gal of water. About one-half of this water is from the Upper Mississippi and one-half from the Missouri, which joins the Mississippi about 15 river miles upstream from Eads Bridge. (Eads Bridge was the second railroad bridge to be built over the Mississippi; the first was at Rock Island, Ill.)

Across the Mississippi, in the area east of the river in southwestern Illinois, there is an area, known locally as the American Bottoms, which lies in portions of Madison, St. Clair, and Monroe counties within the valley bottom, as shown in Fig. 2. The American Bottoms are approximately 30 miles long and have a maximum width of 11 miles; they cover an area of about 175 sq miles. In addition to its proximity to the river, this area is endowed with a plentiful source of 60°F ground water, which, of course, is attractive to industries requiring large volumes of water for process work.

Public Water Supplies

Alton, Ill., lies at the northern extremity of the American Bottoms.

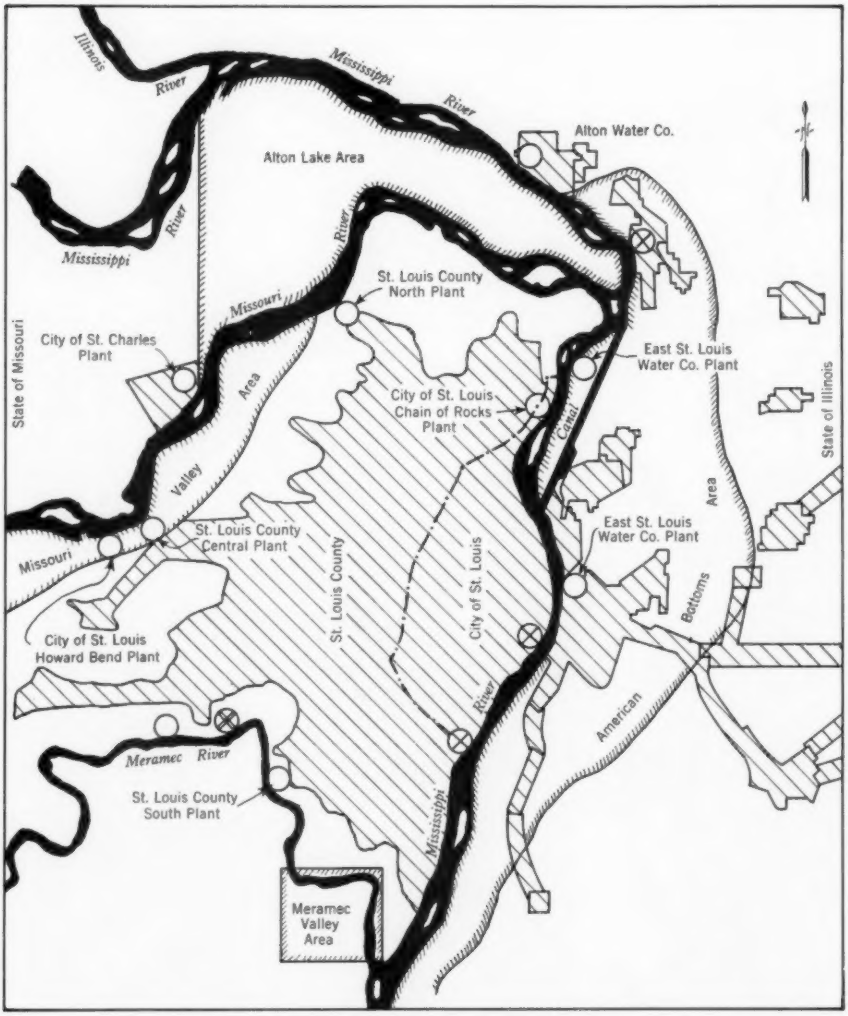

Fig. 1. Water Resources in the St. Louis Metropolitan Area

The white circles indicate surface supply intake stations; white circles with single x *indicate industrial surface supply intake stations; and white circles with double* x *indicate public supply from radial-collector wells. Cross-hatched area indicates areas served by public water supply. Population for the area is nearly 2,000,000, the area's public surface supply consumption is approximately 340 mgd, and the industrial ground water consumption is approximately 123 mgd.*

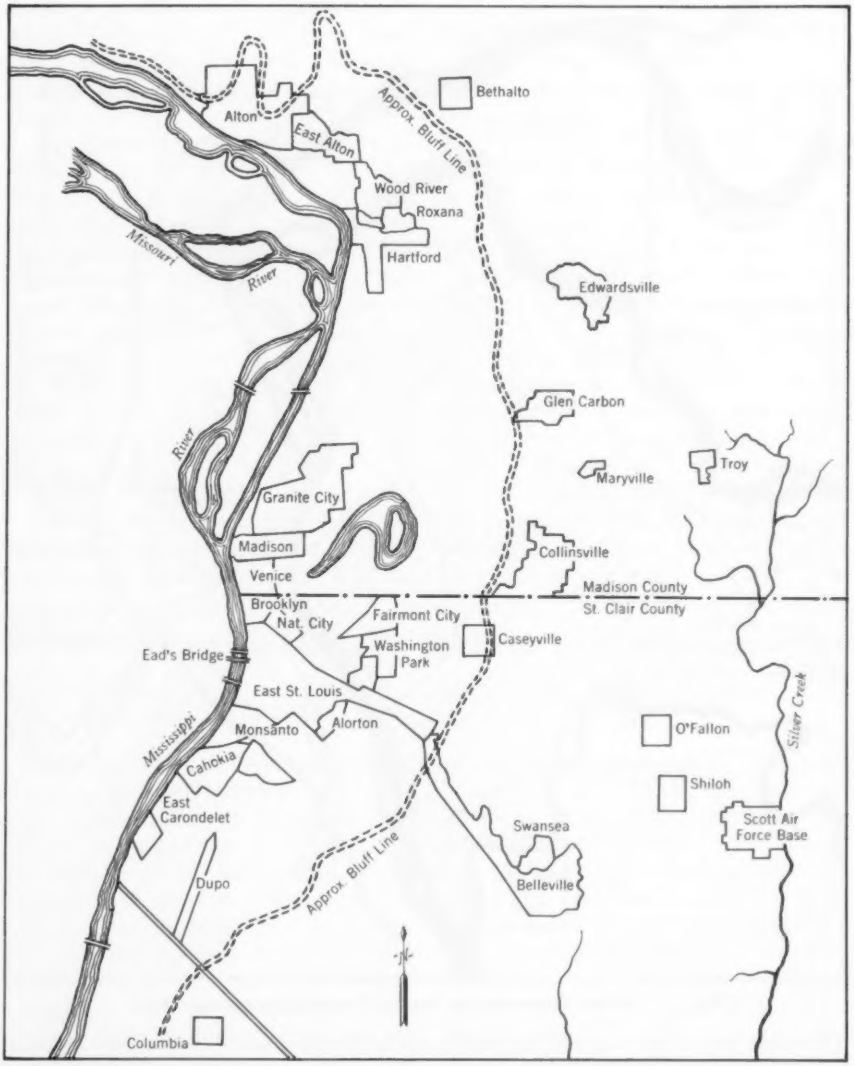

Fig. 2. Cities, Towns, and Municipalities East of the Mississippi River

The Alton Water Company serves Greater Alton, with its population of 65,000, including the villages of Brighton and Godfrey located in the upland area. About 8 mgd from the Mississippi is treated, and delivered by this operation.

Immediately adjacent to Alton is East Alton, with a population of 7,500. The water utility serving the village also supplies a large industrial facility. The daily supply for these users consists of 1 mil gal of ground water and 5 mil gal of treated surface water.

South of East Alton are Wood River, Roxanna, and Hartford, with a combined population of 15,000. The largest of this group is Wood River, a town of 10,500 population, with five wells which deliver an average of 1 mgd to the community. From this bottom area, three wells provide water to Bethalto, located in the upland area and having a population of 2,200.

Several oil refineries at Wood River and Roxanna use large quantities of both surface and well water in their extensive operations. About 15 mgd of surface water and 23 mgd of ground water are used for these operations.

The city of Edwardsville, with a population of 9,000, is located in the upland area. It is served from a ground water plant at Poag, located about 5 miles from the city. Three wells deliver an average of 750,000 gpd of treated water. During 1954, for a period of 30 days, this plant delivered by tank truck 250,000 gal each day to the village of Highland, Ill., when the village supply was practically exhausted because of drought conditions. This required a road haul of 16 miles to furnish the stricken village with sufficient water to meet its essential needs.

Collinsville, a city of 12,000 population, and the villages of Troy (population 1,300), Glen Carbon (1,200), and Maryville (600), are located in the upland area but receive their water supply from wells in the American Bottoms. Collinsville, with four wells, delivers an average of 1.5 mgd.

Directly across the river from St. Louis is the largest city in the area being described. East St. Louis, Ill., with a population of 90,000, has a trading-area population of over 120,000. The East St. Louis and Interurban Water Company serves all the towns and villages shown in Fig. 2, except those previously described, and several additional towns and villages in the upland area. In the American Bottoms area, the cities served include: Nameoki and Granite City (now combined), Madison, Venice, Brooklyn, National City, Fairmont City, Washington Park, Caseyville, East St. Louis, Monsanto, Cahokia, Dupo, East Carondelet, and Columbia. The cities and villages served in the upland area, which is approximately 200 ft higher in elevation, include Belleville, Swansea, O'Fallon, Shiloh, and Scott Air Force Base. During 1955, an average of 36 mgd was delivered into this integrated system from two main pumping and purification plants located in East St. Louis and Granite City, Ill. The source is the Mississippi River.

A considerable concentration of industry prevails in the area. Approximately 6,430 mil gal, or 51 per cent of the total delivery in 1955, was used by industry. In addition to the surface water requirements, much of the industry uses a considerable quantity of ground water. For example, a steel mill uses about 1 mgd from the utility system and 30 mgd of ground water for quenching, cooling, and other purposes. The packing-house district, which includes several large opera-

tions, takes 5 mgd from the utility and uses 15 mgd from wells. A large chemical plant, in addition to 6 mgd from the utility, uses 8 mgd from wells.

The above examples of the combined use of surface water and ground water serve to emphasize that, in the American Bottoms, industry has water not only in abundance but of a character to meet its specific needs. The river provides a limitless source for surface waters, and the use of ground water is now estimated to be well over 100 mgd. The area is capable of producing a still greater yield, if necessary.

Ground Water Recharge

A number of reports have been made on the American Bottoms area. One of the earliest was made by the Illinois State Geological Survey in 1907. Recent surveys include the following:

1. In 1953 an excellent report was made by Jack Bruin and H. F. Smith, of the Illinois State Water Survey (*1*). This report stated:

> Since 1941, there has been no general recession of water levels except within the influence of the three areas of concentrated pumpage and in National City, where some lowering has been caused by pumpage.
>
> The lowering of ground water levels within the zones of influence of heavy pumpage has not caused any serious depletion of the ground water resources.

2. A recent investigation (*2*) by Bergstrom and Walker of the State Geological Survey was completed in 1956. The following is taken from their report:

> The principal means of recharge of ground water in the valley fill are seepage from rainfall and floods, and percolation from the Mississippi River and its tributaries. Rainfall is probably the most important source for the area as a whole, although where heavy pumpage is concentrated near the river the recharge from the river itself is undoubtedly great.
>
> Despite the present heavy industrial ground water consumption, it is likely that much more ground water could be available if industrial expansion takes place in favorable but unexploited areas, particularly near the river where recharge might be induced.

Conclusion

A population of 50,000 in ten towns and villages located in the bottom and upland areas are supplied with ground water from the American Bottoms. More than 300,000 people residing in 26 towns and villages are furnished with surface water.

The area described is one of the great industrial centers of this country, and the abundance of water is one of the principal reasons for this concentration of diversified industry.

Much greater expansion is still possible in the area. In the American Bottoms, in addition to water, there are available all of the facilities necessary for industrial expansion, including land, transportation, and electric power. Also, because the area is at the crossroads of the nation, raw materials and markets for the finished product are comparatively close.

References

1. Bruin, Jack & Smith, H. F. Preliminary Investigation of Ground Water Resources in the American Bottoms. Rept. of Investigation No. 17, State Water Survey Div., Urbana, Ill. (1953).
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City of St. Louis—*John B. Dean*

A paper presented by John B. Dean, Div. Engr. in Charge, Supply & Purifying Sec., St. Louis Water Div., St. Louis, Mo.

Within the city of St. Louis, water is available from the Mississippi River, from wells, and from the mains of the municipally owned water system. Many of the manufacturing plants and available plant sites are located adjacent to the river, which forms the eastern boundary of the city. Plants at these locations can obtain water from the river for use with or without treatment. One utility is now using river water for cooling condensers and a brewery and a paint manufacturer are treating it for use in their plants. The amount of river water available for such purposes is more than could possibly be used.

Wells

Formerly, water was obtained from wells within the city to a much greater extent than at present. There were approximately 40 wells, varying from 200 to 3,880 ft in depth, that were drilled into rock. The water from a large percentage of these wells was so mineralized that it could not be used for drinking. Generally, the shallower wells were more likely to produce fresh water than the deeper ones. Nearly all of the wells in the city have been abandoned, probably because they have become polluted and because the city supplies water at a very reasonable rate.

The temperature of well water in this vicinity is approximately 60°F throughout the year, whereas the temperature of city water varies from 35° to 87°F. In cases where water is needed for cooling, the use of well water during summer months would seem to be desirable, and, for plants located on bottom land, the construction of radial-collector wells or smaller wells cased with precast concrete cylinders might be economically justified.

City Mains

St. Louis obtains water from the Missouri River at Howard Bend and from the Mississippi River at Chain of Rocks. The amount obtainable from the Howard Bend Plant is limited to 120 mgd by the capacity of the two 60-in. coduits conveying water to the city. At this rate of operation there are a raw-water pump and a high-service pump, each of 60-mgd capacity, standing by. The capacity of the Chain of Rocks Plant, with all of the high-service pumps in use and a 35-mgd raw-water pump standing by, would be 220 mgd. The total capacity of the entire system for meeting peak demands is 340 mgd.

A review of the records shows that during the last 15 years there has been a fairly uniform yearly increase of 3 mgd in the average daily consumption. Similarly, from 1941 to 1952, the average annual increase in the maximum daily consumption was approximately 3 mgd, but, between 1952 and 1955, it increased from 213 to 283 mgd. During the fiscal year ending April 1956, it dropped to 269 mgd. This enormous increase in the maximum daily consumption is thought to be caused by excessive use of water for sprinkling and air conditioning during the hot dry weather that occurred during the last 4 years, and this curve may level off in the future. Fortu-

nately, there are two reservoirs in the distribution system which take care of the hourly fluctuations and carry over the peak daily demands to the weekends, when the reservoirs can be filled. After the new high-service pumping station at Chain of Rocks and the new pressure conduits to Baden and Bissell's Point are completed in 1961, there will be a firm capacity of 420 mgd available.

Because the city limits are fixed and most of the space suitable for residential use is built up, increases in demand are expected to result mostly from increased use by manufacturing plants. Data supplied by the City Plan Commission indicates that available factory sites of more than 3 acres each total over 2,000 acres, which fall roughly into three groups as follows: 400 acres in south St. Louis, between Gravois Avenue and the river; 300 acres in the central part of the city, mostly along Mill Creek Valley; and 1,300 acres in north St. Louis, mostly along the river front. By laying mains for short distances, the amount of water required in each of these areas can be readily supplied from existing large mains.

One of the conduits from Howard Bend Plant terminates approximately in the center of the first area, and three 20-in. mains are laid to the vicinity where most of the property is located. Mill Creek Valley, in the second area, is crossed by a 60-in. conduit, a 36-in. main, and two 30-in. mains at points near the factory sites.

The third area can be supplied from the pumping stations at Baden and Bissell's Point, which are in the vicinity of this undeveloped property.

It is the author's opinion that, except in cases of catastrophe, all users of water within the city will be supplied with the amount of water needed.

St. Louis County—*W. Victor Weir*

A paper presented by W. Victor Weir, Pres. and Gen. Mgr., St. Louis County Water Co., University City, Mo.

St. Louis County is almost surrounded by an unlimited supply of good surface water. The Missouri River is on the northwest and north sides and the Meramec River is on the southwest and south. The Mississippi River is the county's eastern boundary north and south of the city of St. Louis. As it passes St. Louis, the Mississippi receives industrial wastes in large quantities from both sides of the river. South of St. Louis, therefore, the river is not suitable for public water supply uses; the sanitary wastes of practically the entire metropolitan area have been dumped, untreated, into the river. Ground water supply exists only in the very western part of the county and in the alluvial plains of the Missouri and Meramec Rivers.

Resources

From the viewpoint of the water works industry, water resources are good. There is ample surface water available and the basic quality is excellent. From the viewpoint of the citizen or local industry, however, the fact that there is a large supply of water within 9 miles of any point in the county matters very little. He is concerned only with the availability of water at his particular location, and

he doesn't care if the Missouri flows $2\frac{3}{4}$, 41, or 580 bil gal daily, which are the minimum, average, and maximum flow figures. He wants to know whether he can get 10 gpm at his house or 1,000 gpm at his factory when he wants water. Those are the figures and the water resources that interest him.

In talking about water in St. Louis County, therefore, the most important questions concern the resources of the water utility—finances, personnel, and policy. Generally, the same situation prevails elsewhere in the country. The Missouri River, or Lake Michigan, or Lake Erie, or the Hudson River, may be within a gunshot, but the resources which are responsible for supplying water service are those of the utility serving the area.

History

St. Louis County, prior to 1875, was an area of 534 sq miles. In 1875, by a special act of the legislature, the city of St. Louis withdrew from the county. The limits of the city then embraced all the rural area which was thought necessary for the ultimate growth of the city. This area, covering 61 sq miles, was practically saturated in the 1930's, and most of the area's population growth took place in the county after that time.

There are now 96 incorporated municipalities in the county. The population, which was 274,000 in 1940 and 406,000 in 1950, is now estimated at 600,000. There were only twelve cities in the United States with populations exceeding 600,000 in 1950. Although the county comprises only about one-third of 1 per cent of the national population, the housing starts in St. Louis County for several years have approximated 1 per cent of national housing starts. These have been principally single-family dwellings; despite a surge of industrial development in recent years, the area is still predominantly residential.

Terrain in the county is very rolling. The ridges between the three rivers are 250–330 ft higher than the river bottoms and the area is broken by the valleys of numerous creeks. Population growth has taken place over the entire area, however, with no regard to elevation.

The principal water utility in the county is the St. Louis County Water Company. It supplies directly more than 126,000 customers and provides wholesale supplies to the cities of Webster Groves and Florissant and to one water district. These three systems in turn supply more than 14,000 customers. Webster Groves obtains a part of its supply from St. Louis. Kirkwood, the only large city with its own plant near the Meramec River, gets part of its peak requirements from the St. Louis County Water Company.

The rapid population growth since the last national census has added 190,000 to the county population. In 1950, there were only 50 cities in the nation with a population larger than the population increase, itself, in St. Louis County for the last 6 years. It is as if all the people in Salt Lake City or Des Moines or Hartford or Grand Rapids or Nashville had left their homes and moved into St. Louis County after 1950.

These new citizens did not all connect to existing mains. In the last 6 years the St. Louis County Water Company has installed 535 miles of pipe, 4,000 fire hydrants, and has increased its plant capacity by 72 mgd. It has spent nearly $23,000,000 while getting 47,000 new customers. This is $490 per new customer.

In 1952 the country moved into a period of prosperity. That same year, however, marked the beginning of a severe drought in the area, which lasted for 3 years.

In spite of a 20 per cent increase in water rates (the first increase in the history of the St. Louis County Water Company), the customers wanted more water per customer than they had ever wanted before. The result was that the water resources, as far as the customers were concerned, were not what had been expected. The peak day was 24 per cent greater than the peak day in 1951. It was necessary to curtail sprinkling of lawns in the afternoons of hot days. By the summer of 1953, capacity of plant and mains had been increased by 23 per cent, but so had demand. Again, sprinkling restrictions were necessary for a short period. By the summer of 1954, capacity was increased by 35 per cent over the 1953 maximum day. This time, however, demand lagged behind slightly, increasing by only 33 per cent. Sprinkling restrictions were invoked in three cities having their own distribution systems, but not in the county system. In 1955 no difficulty was experienced. A 36-mgd plant added in 1955 was not actually needed. In 1956, a 15-mgd plant unit will be added to the St. Louis County Water Company. No shortages are anticipated, even though the very worst of drought conditions occur.

County Water System Resources

As has been stated, the water resources of the residents depend upon the resources of the utility. Some of those resources at the St. Louis County Water Company are as follows:

Adequate personnel. Of 385 employees on the payroll, approximately 220 are involved in construction work. The other 165 employees are engaged in operation and maintenance.

Alert management. It is not enough for management to keep up with the problems of today; it must keep ahead of the problems of tomorrow. This is similar to the game of chess, in which a player is not concerned only with his opponent's last move, but also with what his opponent will do in the next couple of moves.

Ability to act quickly. Although the necessity of spending up to $75,000,000 in the next 15 years can be foreseen, it is not known just where the money will be used in the service area, which now covers more than 200 sq miles, and which may cover 300 sq miles 15 years from now. Subdividers move quickly and independently, and the utility has to follow them with the facilities to deliver water. Fortunately, there is no waiting for elections on bond issues, and there is no problem of getting favorable commitments for large sums when no emergency faces the citizens.

Adequate financing. Expansion today costs much more than it did 10 or 15 years ago. The St. Louis County Water Company expects that it may have to spend as much as $75,000,000 between now and 1970, when the county population may exceed 1,000,000. Money must come from bank loans, bonds, and stock. The water company's credit is good, however, and the stockholder understands the problems involved in providing water facilities for a rapidly expanding community. The need for money for improvements has meant that no dividends have been paid in 15 years. This has been a prime factor in financing the system's growth along with, and ahead of, the population.

Financial strength. Many water customers will say that an eagerness to keep a utility financially strong is not a resource, because it requires asking for higher water rates when they are needed. It may not be popular to ask for higher rates, but it is necessary if good, rather than fair or poor, service is to be rendered. Enlightened customers agree that the difference between good and poor service is worth more than 1–3 cents a day per family. The fortitude needed to ask for an unpopular rate increase so the utility will be in position to render good service during the next summer's drought, or during some future emergency, is indeed a resource which the community deserves.

Conclusion

As a whole, the water resources available to utilities in St. Louis County are ample. The resources available to process and deliver the water to the customers, now and in the future, are excellent.

St. Louis Metropolitan Area—*William B. Schworm*

A paper presented by William B. Schworm, Sr. Chem. Engr., Howard Bend Station, St. Louis Water Div., Chesterfield, Mo.

On a visit to St. Louis in the latter part of the 19th century, Mark Twain remarked that the reason the hotels of the city furnished a half dozen cakes of soap with each bath was because once the soap slipped out of your hand there was no use looking for it. Some improvements have been made in quality since then.

Surface Supplies

Most of the water purified in the plants of the metropolitan area is drawn from surface supplies. Because the city is located on two of the great rivers of the world, quantity is not a problem. These rivers have wide seasonal variation in turbidity, color, and dissolved solids, and, considering the vast drainage area, the raw-water quality is quite unpredictable. This complicates the operation of the plants of the vicinity, requiring a close supervision of all phases. The operators' jobs are therefore both fascinating and challenging.

The quality of the Missouri and Mississippi Rivers is changing. This may be because of dams, the prolonged drought, and other factors. The Missouri, particularly, has shown a marked reduction in suspended load. It and the Mississippi are carrying an increased burden of pollution, both sanitary and industrial, which was not apparent a few years ago. The pollution is particularly noticeable during the winter months, when stages are lowest and ice is present.

The presence of pollution means an increase in the use of expensive chemicals, such as activated carbon, and larger doses of chlorine and chlorine dioxide. This, of course, increases the cost of purification for the removal of tastes and odors, making them out of all proportion to the cost of softening and turbidity removal.

Water Plants

Although the plants of the vicinity are scattered, they are all using the

same water, with the exception of the Kirkwood Plant, the Valley Park Plant, and the South Plant of the St. Louis County Water Company, all of which are located on the Meramec River. This is a relatively small stream of limited drainage area which is quite clear most of the time, but easily upset by local rainstorms.

All of the plants in this area use chemicals to coagulate or soften, or both. These may consist of various combinations of lime, ferrous sulfate, ferric sulfate, and aluminum sulfate.

Although the Chain of Rocks Plant is located on the Mississippi River, it is only a short distance below the confluence of the two rivers, the waters of which are not there thoroughly mixed. The plant therefore treats water that comes mainly from the Missouri. Treatment consists of softening with lime and coagulation with ferrous sulfate and alum. The quality of the effluent from this plant is about the same as from the two St. Louis County plants and the St. Louis City Howard Bend plant, which are all located on the Missouri River.

A study of the results of mineral analysis from the four plants will show similarities in both raw and finished water. All reduce the hardness by about 50 per cent with lime, producing a water of 100 ppm hardness, as calcium carbonate.

The two plants of the East St. Louis and Interurban Water Company, located on the eastern bank of the Mississippi, face a far more complex problem. Here, industrial pollution not present on the western side of the river requires formidable chemical charges to produce a satisfactory effluent. Treatment consists of heavy doses of alum with some lime, followed by activated carbon and chlorine dioxide for taste and odor removal.

It is obvious, of course, that all the plants use chlorine and ammonia in various combinations for disinfection.

Of the three plants on the Meramec River, only two are presently in operation. The new St. Louis County Water Company's South Plant will not be in full operation until later in 1956. Of the two that are in operation, the larger Kirkwood Plant is the more important. Drawing its supply from a radial-collector well, it reduces the initial hardness of 260 ppm to about 120 ppm. The initial turbidity is but 3 ppm, whereas water taken directly from the river would run considerably higher.

The bacteriological quality of the effluent from all plants in the area which produce water for domestic use surpasses the USPHS standards for water for interstate use (*1*). This is not always easily achieved. During the flood of 1951 some of the cities in the area had their clear basins and gravity flowlines under several feet of river, and, as most of the plants are located in the floodplain, precautions must be taken to insure that such conditions are not repeated.

Industrial Plants

There are three industrial water plants in the metropolitan area which have been designed for the specific purpose of furnishing process water, and are large enough for mention.

The 7-mgd Anheuser-Busch plant produces an effluent which is low in hardness and carries about 3.5 ppm of residual chlorine. It, like East St. Louis, must contend with pollution caused by sewage from the city and wastes from various manufacturing

plants located immediately north. Raw-water color sometimes runs as high as 60 ppm for short periods. The filtered water is used for all purposes except for beer making. In washing operations it is further fortified with chlorine to 7 ppm residual.

The Wood River and National Lead Company plants use water mainly for cooling and process operation. The water from the 12-mgd National Lead plant is used to wash titanium dioxide pigments free from iron and other impurities which might detract from whiteness. Treatment with lime, ferric sulfate, and activated silica produces a water with a turbidity of less than 5 ppm.

Ground Water

The limitations of quantity and quality of water from wells located on the western side of the Mississippi River make ground water unimportant as a resource. These waters are highly mineralized and most of them contain considerable quantities of hydrogen sulfide. One of them, because of its unique history, is worthy of mention. This is the Belcher well located at Main and O'Fallon Streets, owned by the Belcher Hotel Company. It was originally drilled in 1849 to furnish water for a sugar refinery, but, when found to be unsuitable for this use, was thought to have curative properties and is now used for baths. It has a depth of 2,199 ft and cost $10,000 when completed in 1854.

The chemical quality of the wells located in the American Bottoms is quite variable from place to place. In general, these wells are usually less than 125 ft deep. The waters are quite hard, one running 3,000 ppm. Iron varies from trace to 120 ppm, and averages 9 ppm.

Reference

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Centralized Distribution System Control in the Washington Suburban Sanitary District

John M. Jester and John W. Henderson

A paper presented on May 9, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by John M. Jester, Maint. & Operation Div. Engr., and John W. Henderson, Elec. Engr., both of the Washington Suburban San. Com., Hyattsville, Md.

THE Washington Suburban Sanitary District covers approximately 302 sq miles of land in Maryland's Prince Georges and Montgomery counties adjacent to and surrounding the District of Columbia. The present estimated population served in the district is slightly under 500,000. The main source of water supply for the area served is Patuxent River, a tributary of Chesapeake Bay, across which have been constructed two slab-and-buttress dams. The uppermost of these, the Brighton dam, was completed in 1942, while the Rocky Gorge dam, which is located approximately 11 miles downstream from the Brighton dam, was completed in 1954. The drainage area above the two dams, which impound a total of 12.5 bil gal of water at normal pond level, is 132 sq miles. The normal pond level of the Rocky Gorge lake is at el 285. Below this has been constructed a raw-water pumping station with a pump capacity of 94 mgd. The pumps, which are of the centrifugal type and driven by electric motor or water-powered turbine, take suction at el 165, thus having a positive suction head of 120 ft at full pond level. During periods of excess flow in the river, 51 mil gal per day can be pumped to the filtration plant with three centrifugal pumps directly connected to water-powered turbines totaling 1,950 hp.

Patuxent Plant

From the Rocky Gorge pumping station the raw water is lifted to the Patuxent filter plant at el 425. This plant, which has a capacity of 65 mgd, is of the Morse type, consisting of four complete filter and sedimentation units, a head house containing the chemical feed equipment, elevated wash water storage, modern laboratory, office space, and a centralized control panel. Filtered-water storage and a high-lift pumping station complete the plant which, with the exception of the high-lift pumping station, is of all welded-steel construction. The high-lift pumping station is of reinforced concrete.

The Patuxent plant, the first unit of which was placed in operation in 1944, is unique not only because of its type of construction, but because of the arrangement, which permits the addition of units as the needs of the community require.

At the beginning of 1956, the Washington Suburban Sanitary Commission had in service 1,294 miles of water mains to which were connected some 97,858 water services. In 1955 the

average water consumption was 35.083 mgd.

Although filtered water is stored at the Patuxent plant to maximum el 415, the topography of the areas to be served is such that in order to maintain satisfactory operating pressures, booster stations are necessary to serve some areas, requiring lifts up to el 660, while pressure-reducing installations are required for mains laid near sea level.

Early in 1945 the commission had in operation one booster pumping station (the Capitol Heights station) which was manually operated and required four full-time attendants. The water distribution system was expanding rapidly and the need for additional booster pumping stations was becoming apparent. A lack of qualified personnel, together with increasing operating costs and the need for better operating control, led to a study for the purpose of making this station fully automatic in operation with supervisory control at a central location. What was then the commission's main office building, located about 6 miles away, was used for a control center. The changeover of the pumping itself from manual operation to automatic control was not very difficult, for there was available at the time suitable automatic equipment for electric-motor operation, and the authors had gained considerable experience with solenoid pilot valves for pump discharge valve control. In fact, the first unit of the Patuxent plant, which had been placed in operation only a year earlier, contained a central control console for the operation of the control valves for its six filters, rate controllers, chemical feeders, wash water pumps, and alarm devices. The immediate problems were:

1. What data should be transmitted to the control center for adequate control of the station?

2. How was this information to be transmitted and received at the control center?

In solving the first problem it was decided that only such pertinent information as pump suction pressures, pump discharge pressures, and the air temperature in the station would be required. The pressure available on the suction side of the pump would also show the amount of water available in storage on the low side of the unit, while the pressure readings on the discharge or high side of the unit would show whether or not the station was operating, and the number of pumping units in operation. This would be indicated by increments in pressure rise as each unit went into operation. Inside air temperature was considered important because of the possibility of failure of the heating system during the colder months of the year.

In solving the second problem a system was worked out whereby the desired information is transmitted over a single pair of wires and, at the receiving end of the circuit, the signals are sorted and directed to the proper receivers for recording.

The transmitters and receivers are conventional units which operate on the principle of translating the quantities to be measured into cyclic time impulses. A program timer, consisting of a series of switches opened and closed by revolving cams driven by a synchronous motor, is located at each end of a leased telephone circuit. The timer at the receiving end runs continuously, while that at the transmitting end stops at the end of each cycle, which is a few seconds less than the cycle of the other. When the receiving program timer completes its cycle, it sends a signal which starts the

motor driving the transmitting program timer, thus keeping the corresponding cams and switches of both timers synchronized. When the corresponding pairs of switches are closed, a timed impulse indicating the quantity being measured by the transmitter is sent to the proper receiver, which then records the quantity on a chart. A more detailed description of the system is contained in a previous article by the authors (*1*).

Capitol Heights Booster Station

The four pumping units installed in the Capitol Heights station are controlled by a device which has two independent pressure elements of the spring-loaded bellows type for each pump. The low-pressure element starts the pump motor and opens the hydraulically operated discharge valve; the high-pressure element closes the discharge valve. When this valve reaches the closed position, a limit switch opens and stops the motor. After the equipment had been put in operation, it was found that the surges caused by bringing another pump on the line caused the high-pressure element of the running pump to function, thus shutting down the unit. To prevent this action time delay relays were installed in those circuits energized by the high-pressure elements, and are functioning in a satisfactory manner.

As the filtered-water storage elevation at the Patuxent plant is 415 ft and the controlling elevation of the Montgomery County main pressure zone is 495 ft, water is pumped from storage at the Patuxent booster pumping station by three electric motor–driven centrifugal pumps having a maximum combined capacity of 56 mgd. Each unit is driven by a 600-hp dual-speed electric motor operating at 885 or 1,180 rpm. Although these units can be operated from the control panels in the station they are normally operated by remote control from the central panel located in the Patuxent filter plant.

For this purpose a supervisory control system, like that used by electric utilities for remote control of circuit breakers and hydraulic turbines, was first investigated, but the cost was found to be too high. Then a preliminary design was made of an audio tone system and an estimate of the cost thereof was obtained from a manufacturer of electronic equipment. The estimated cost, although less than the commercial supervisory-control system, was also deemed excessive. Finally a system, using rotating program timers, similar to that described for the Capitol Heights station, was designed and installed. This system transmits twenty control functions from the main control panel in the filter plant to the pumping station and 21 confirming signals from the pumping station to the control panel. The control functions for each of the three existing two-speed pumps and a future pump are: run low speed; run high speed; emergency stop; open discharge valve; close discharge valve. (The pumps are normally stopped by means of a limit switch on the discharge valve.) The confirming signals are colored lights on the control panel, which indicate that the pumps are not running or are operating on either low or high speed; that the discharge valves are opened or closed; and that the program timer in the pumping station is revolving. The timer in the filter plant runs continuously, completing its cycle in 35 sec. The timer in the pumping station makes its cycle in 30 sec, then stops until started again by a signal from the other at the beginning of each cycle. The signals are transmitted by means

of twelve pairs of conductors in a cable similar to that used for telephones. The other pairs in the cable are used for intercommunication and the transmission of flow and pressure data. A confirming signal is obtained within 30 sec after a control function is initiated. The failure of the reduction gearing of the motors which drive the timers has been the principal source of trouble with the system. It was found that the reduction gearing had fiber gears meshing with steel, and that the failure was apparently due to lack of proper lubrication. The manufacturer of the motors finally advised the proper grade, amount, and frequency of lubrication. Pressure and flow data are also transmitted to the main control panel where they are recorded.

Montgomery Booster Station

From the Montgomery main zone, maximum water el 495, it is necessary to lift the water to el 650 to serve the Montgomery first high-pressure zone. A reinforced-concrete structure located between two 4-mil gal steel reservoirs houses two electric motor–driven centrifugal pumps having a combined capacity of 2.6 mgd. A third steel reservoir of 15-mil gal capacity is currently under construction, and a third pumping unit, which will increase the capacity of this station to 5.6 mgd, will be in operation in time to meet the requirements of the approaching summer demand.

The control equipment in this station consists of a dual-pump device similar to that described for the Capitol Heights station. The time delay relays are omitted as they are not required at the present time. When the new unit is installed, a three-pump controller with built-in time delay relays will be installed in place of the existing one. The telemetering equipment is identical with that at the Capitol Heights station. At the present time, operating data from this station are transmitted to and recorded at a secondary control center, from which point they are relayed by land line to the main control center at the Patuxent plant. Present plans call for the abandonment of this secondary control center and the relocation of its receiving and recording equipment on the main panel at the main control center.

From the Montgomery first high-pressure zone, maximum water el 650, it is necessary, because of line friction losses, to boost the pressure in the main feeding Gaithersburg and Washington Grove, which are in a remotely located area. For this purpose the commission has in operation a small water booster station of masonry construction with two electric motor–driven centrifugal pumps having a total capacity of 2.2 mgd. The sequence in control operation of this station differs radically from that of the commission's other booster stations in that, when a pump goes into operation, the motor starts against no load and, when running speed is obtained, the pump discharge valve opens. In the normal operation of the commission's booster stations this sequence is reversed to shut the unit down for the control of water hammer; that is, the pump discharge valve closes first before the motor is deenergized. When it was attempted to place this station in operation following this procedure, a severe water hammer developed on the pump suction main. This condition continued to develop in spite of the fact that the pump discharge valve was closed for a period of some 300 sec, which was as slow as the discharge valves could be operated. The devel-

opment of this surge was caused by the fact that the suction feed serving this station was approximately 46,000 ft long, without any branch lines. Several methods of controlling this condition were being considered when it was suggested that the pump be shut off with the discharge valve in a wide-open position, the theory being that as the surge developed in the suction line, which exceeded the pressure on the discharge side of the pump, the shock wave would be dissipated through the check valve located on the discharge side of the unit. With somewhat more hope than confidence, recording equipment was set up in the station and along the pump suction line, and the unit was shut off. The result was that the pump came to rest with practically no surge occurring in either the pump suction or discharge lines.

The pumps in this station are controlled by a dual-pump control device similar to those previously described, except that time delays in the starting and stopping circuits are incorporated in the equipment. The high-pressure element, instead of closing the discharge valve, stops the motor and energizes a time delay relay, which closes the discharge valve after the surge in the suction line has dissipated. The telemetering equipment is identical with that in the Capitol Heights station. Operating pressures and flow data from this station are transmitted to and recorded on the central control panel at the Patuxent plant.

Prince Georges Plants

The filtered-water storage at maximum el 415 at the Patuxent plant is sufficient to serve by gravity the Prince Georges first high-pressure zone, which also includes an area in southern Howard County. Although Howard County is not a part of the sanitary district, by an agreement with the Howard County Metropolitan Commission the Washington Suburban Sanitary Commission furnishes the water supply for, and maintains and operates, the system in this part of Howard County. To serve the Prince Georges main zone, which has a controlling elevation of 320 ft, pressure-reducing installations have been made on each of the two main feeder mains, of 30- and 42-in. diameter respectively, feeding this zone from the Patuxent plant. The pressure-reducing installation on the 42-in. feeder main is installed at el 128 and reduces the water pressure from 120 psi on the high-pressure side to 78 psi on the low-pressure side. For this purpose there are installed two 24-in. pressure-reducing and regulating valves operated hydraulically. The original installation has electric remote controls located on the central control panel at the Patuxent plant. The pressure-reducing installation on the 30-in. feeder main installed at el 141 reduces the water pressure from 115 psi on the high side to 73 psi on the low side. Pressure-reducing and regulating valves of 12 and 20 in., hydraulically operated with spring-loaded pilot valves, are in use for this service. The operation of this unit, which was installed in 1944 without remote control, is for the 12-in. regulating valve to handle flow demand up to its maximum capacity, at which point the 20-in. valve takes over. Although this type of installation proved satisfactory as long as the distribution-system demands on the low-pressure side are reasonably constant, it was found that as seasonal demands changed, the desired control could not be maintained without sending personnel to the installation for the purpose of adjusting the controls in order to change the range settings. Actually

this procedure required the services of two men, as the commission does not permit a man to enter a vault without a second man in attendance on the surface. In addition to the expense involved in such operation, delays frequently occur in locating and dispatching the proper personnel required.

When a similar installation was required for the 42-in. main feeder line several years ago, several methods of remote control for the pressure-reducing and regulating valves were studied, and it was finally decided that the pilot control should be motor operated by remote control from the central control panel at the Patuxent plant. To accomplish this, one of the pressure-reducing valves is adjustable by means of a reversible limit-torque motor, which varies the tension of the springs in the pilot valve. Revolving program timers in the valve vault and at the filter plant, like those used in the telemetering systems previously mentioned, enable the operator at the filter plant to control the valve by means of holding a switch on the central control panel in either the "raise" or "lower" position until the gages on the panel indicate the desired pressures. The cycle of the timer is 180 sec. The contacts of the cam-operated switches in the "raise" and "lower" circuits are closed simultaneously for 40 sec. Indicating lights on the panel show when the control circuits are available. Operating pressures are transmitted to the central control panel where they are received and recorded.

To serve the Prince Georges second high zone, water from the Prince Georges main zone is pumped to el 450 through the Hill Road booster station. This station contains three electric motor–driven centrifugal-pumping units having a total capacity of 13 mgd, and secondary chlorination apparatus. The control equipment in the Hill Road station differs from that previously described. The pressure-actuated equipment has bellows which are loaded, through linkage, by a pendulum, and synchronous-motor time delays in both the starting and stopping circuits. The pumps are started by the low-pressure circuits. The high-pressure circuits close the discharge valves, which in turn stop the pumps by means of limit switches. The telemetering equipment is similar to that previously described, except that four quantities are transmitted to the central control panel: water elevation in the Hill Road reservoir, from which the pumps take suction; discharge pressure; flow in the discharge line; and temperature in the chlorinator room.

Deep Well Pumping

To serve a small area located along the Potomac River and (at the time it was acquired) some distance from the commission's nearest main water supply, a deep well source of water supply was developed. This supply consists of two gravel-packed wells, approximately 600 ft deep and 1,300 ft apart, having a combined capacity of of 925 gpm. Water from the two wells is pumped into a reinforced-concrete receiving reservoir from which two electrically driven centrifugal pumps, having a combined capacity of 1.4 mgd against a 278-ft head, take suction. Chlorination of the pump suction is accomplished by use of a gas-feed chlorinator automatically controlled by means of an orifice plate and converter. High-lift pump operation is controlled by means of a "tank control" installation, while the deep well pumps are controlled by electronic relays, the probes of which are installed in the receiving reservoir. Operating cir-

cuits are so arranged that, if the high-lift pumps deplete the receiving reservoir, they will be automatically shut down until sufficient water is again available for operation. Operating data for this installation are transmitted to the control center for observation and recording.

Operating Troubles

Operating troubles encountered with the telemetering systems during the past 10 years have been of a relatively minor nature and largely in connection with the program timers, involving gear motor failures, contact spring breakage, and cam wear. Equipment repairs are made by the commission's own crews, and the necessary repairs are usually made in a matter of a few hours. Leased line facilities, with the exception of one pair consisting of open wires on insulators, which seem to fail even in foggy weather, have been very satisfactory.

It has been found that the pressure-actuated controller used in the Hill Road station holds its adjustment, while the spring-loaded ones require periodic readjustment, as temperature variations seem to affect the spring tension.

During one of the hurricanes which hit Maryland in August 1955, the area in the vicinity of the high-lift pumping station and receiving reservoir for the previously described well supply became inundated, and the station was surrounded by flood waters. In fact, water rose to a depth of 2 ft at the entrance to the station, and access to the station was impossible without causing it to flood. A two-way radio crew was standing by, observing this station as the flood waters rose, and although the author and his associates passed some rather unpleasant hours wondering whether or not the entrance doors would fail as the water kept rising, or whether leakage past these doors would exceed the sump pump capacity, they were able to observe the operations of this station from the control room during the entire storm.

In 1945, when the installation of multiple telemetering was first contemplated, a method of transmitting the signals simultaneously over leased telephone circuits by means of audio tones of different frequencies was studied. The terminal transmitting and receiving equipment was designed to be used in connection with conventional time impulse transmitting and receiving instruments. At that time the cost of a leased telephone circuit was 75 cents per ¼ mile of line. It was learned, however, that the telephone company would permit only one signal at a time over a leased line. The signal had to be direct current. If simultaneous a-c signals were transmitted, each signal frequency would be considered to be a channel, with a charge of 75 cents per ¼ mile for each channel. This caused abandonment of the audio tone system and its replacement with the rotating-program timer system. Recently, the telephone company has changed its regulations and now permits simultaneous audio frequency signals to be transmitted. A long-established manufacturer of electronic equipment has put on the market terminal equipment consisting of tone generators and frequency-selective receivers for telemetering, remote control, and signaling. Twenty simultaneous signals can be transmitted over an average leased line circuit. The Washington Suburban Sanitary Commission is having such a system installed in connection with a large sewage-pumping station now under construction.

Automatic Control of Booster Stations in the Los Angeles System

Marvin H. Owen

A paper presented on May 9, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by Marvin H. Owen, Sr. Mech. Engr., Dept. of Water & Power, Los Angeles, Calif.

THE phenomenal growth of the city of Los Angeles, particularly in the last 10 years, has resulted in a boom in real estate development of new residential tracts. Some of these new tracts are located in hilly areas which, in the past, have been considered uneconomical to develop. Because of the influx of more than 435,000 people into Los Angeles in the last 10 years, however, the situation has changed. Many precipitous mountainsides have been cut up by giant earth-moving equipment, and the ground subdivided into new sites. In other places, shelves have been carved out of the mountainsides and houses built upon them. Even private roads have had to be built to give access to some of these residences. As a result of this constant population growth and increased number of high-elevation developments, the demand for secondary pumping in these areas has been increasing at an accelerated rate. The recent additions to the area supplied by pumping increased the total demand far beyond expectations. The city of Los Angeles, sprawling over 452.6 sq miles in area, requires a constant average flow of 598.8 cfs, or 387 mgd, throughout the year, to supply the water demand of its citizens, with peak 24-hr periods reaching 756 mgd. Unlike most major cities in the United States, Los Angeles is located in diversified terrain. Domestic water is supplied from sea level to el 2,440, all within the city limits. To supply the hilly areas, more than 23,100 hp are used. This power requirement is divided among 53 secondary pumping plants ranging in size from 10 to 3,300 hp.

The first attempt at secondary pumping was by means of a water wheel installed in the Los Angeles River in 1860. The following winter the floods removed this project. After being rebuilt, it was once more washed out, and finally was abandoned. The next pumping plant, installed in 1893, used a 100-gpm triplex pump driven by a leather belt from a Pelton water wheel. The first major pumping-plant installation employed a Snow cross-compound steam pumping engine which went into operation in 1904. Its capacity was 5 mgd against a head of 300 ft. All of the early plants were steam engine–driven and, of course, manually operated.

In 1925 the first attempt at automation was made without great success, although some of the equipment involved is still in use today. This plant consisted of five wells, with 75-hp motor–driven pumps, discharging into

a sump of about 800,000-gal capacity. The well pumps were controlled by float-actuated switches in the sump to maintain the water level at a minimum of 70 per cent of sump capacity. On the roof of the sump, two booster pumps were installed. These were started and stopped by a pressure switch which closed and opened the starter circuit. One big problem of this plant was the fact that the booster pumps operated on a negative suction head with 6–9 in. of vacuum, so a priming device consisting of a hydraulic ejector was installed. This proved to be costly because of the excessive use of operating water from the discharge side of the pump. The ejector water would build up the sump level, even to the point of overflowing.

The water system of the Los Angeles Department of Water and Power has always considered the high reliability of its water supply one of the prime objectives. This has been reflected in the basic design of all the pumping plants which, wherever possible, incorporate either elevated or pneumatic water storage to provide a reserve for peak demands, power failure, or equipment breakdown. In addition to this, the possibility of service interruption is minimized by the installation of spare units in each plant. These pumps are available for immediate operation in the event of failure of the operating unit. In vital locations, where the pumping plant supplies an extended area without adequate storage or where power lines cross high fire hazard brush areas, internal-combustion engine–driven pumps are used for standby in event of power failure. Equipment for these installations is chosen conservatively to operate well below its maximum capacity. The wisdom of this basic philosophy of design has been borne out by the many years of trouble-free service received by the secondary pump areas.

Method of Operation

Pumping-plant installation in the Los Angeles water system may be divided into three categories: [1] manually operated plants with 24-hr operator attendance; [2] semi-automatic plants; and [3] fully automatic plants.

Manually Operated Plants

The number of manually operated plants in the Los Angeles system is fairly small. Each such plant is a carryover from the days preceding the automatic pumping-plant development. As a rule, these plants were originally equipped with steam engine–driven compressors for air-lift wells and steam-driven pumping engines, or a diesel-driven generator which supplied power for a number of wells in the vicinity, as well as for the booster pumps used for pumping the well water into the distribution system. All of these plants have been converted to operation from power line energy, and even though the units are as large as 1,200 hp, thought has been given to converting them to fully automatic operation.

Semiautomatic Plants

As a rule, existing semiautomatic plants are small in size and consist of two units, the first of which is either started manually or by time switch and generally shut off by a pressure-activated mercoid switch. Sometimes the first unit in a plant is started by means of a time switch and the second unit manually, if the line pressure falls excessively. Experience with these plants has not been very satisfactory, as the operator has to anticipate the

demand in the system in order to start and stop the pumps at the proper time of day. Because of the lack of fully automatic control, the operator sometimes has to remain on duty to assure proper operation during a heavy pumping period.

Fully Automatic Plants

Fully automatic units comprise about 90 per cent of all plants now in operation in the Los Angeles water system. The automatic features of these plants have been under development for a number of years and have reached a high state of reliability. The fully automatic pumping plants vary in size from 10-hp induction motor–driven units to 600-hp, 2,300-v, synchronous motor–driven units. The number of units per plant varies, the smallest plant having two and the largest as many as six. In some instances, in addition to electric-driven units, there are also gasoline or diesel standby units which are either coupled through a clutch to one of the electric-driven pumps or operate independent pumps.

Automatic Control System

The fully automatic pumping plant may be operated either automatically or manually, depending on the setting of the control selector switch on the relay panel. When the switch is in the manual position, the automatic selectors are completely bypassed and pump stopping and starting is performed manually. The following is a description of the control system of operation.

Manual operation. Pumps are started by pressing in the manual-start push button of the desired pump and holding it until the hold circuit is completed by the appropriate relays energizing the holding coil through a contact on the pump discharge check valve. Starting of the pump motor deenergizes a green pilot standby light and lights a red pilot light showing that the unit is on the line. To stop the unit, the corresponding "stop" push button is depressed, interrupting the holding circuit and stopping the motor, simultaneously extinguishing the red light and lighting the green light.

Automatic operation. For starting, a drop in the discharge line pressure closes a pressure-actuated starting switch and initiates a time-delay sequence. At the end of the time delay, a selector circuit becomes energized selecting the first available pump and starting its motor; after the motor is started, another timing circuit comes into action, measuring the lapse interval until the predetermined time when the pump has to start pumping water. If the pump is in good order and begins to pump within a preset time, the contact on the discharge check valve completes the circuit for holding relays and deenergizes the timing circuit. The pump is now on the line operating in normal fashion. If the pump does not start pumping during the preset lapse of time, either because of malfunction of the pump or because the motor is inoperative, the timing element energizes another circuit which deenergizes the motor starter, blocks a selector switch to prevent that motor from being started again, and at the same time lights an amber pilot light to indicate to the plant operator that the unit is locked out. The timing device, at the end of this operation, resets itself and begins another selection sequence to place a second pump on the line as described above. When a unit is being placed on the line and begins

to pump, the pressure-actuated starting switch continues to check the line pressure after the pump is running. If the line pressure is not increased sufficiently by the operation of one pump, the automatic selection cycle is initiated, and after going through the time delay as previously described, the second pump is placed on the line in parallel with the first pump. This checking of line pressure for sufficient increase continues; if the placing of the second unit on the line does not bring the pressure up adequately, the cycle is repeated with the third unit and again with the fourth unit, until either the line pressure comes up or all the available pumps are placed in operation simultaneously.

The automatic control system selects pumps in rotation. If, for example, the No. 1 pump completes the run and shuts down, on the next cycle the No. 2 unit will start first, and so forth. If for some reason one of the pumps becomes locked out, the selector system skips over that unit and selects the next available pump, placing it on the line when required. The system also incorporates an "antihunting" provision to prevent the selector from going into continuous operation, searching for a pump, when all available pumps are either in operation or locked out.

The pumps are automatically stopped by the discharge line pressure increase to a preset value. When either of these conditions occurs, suitable pressure-actuated switches energize an automatic selection circuit which takes off the pump in operation without any time delay. If there is more than one pump running at the beginning of a stopping cycle, the automatic controls take off the pump which came on the system first without time delay, then the second pump after a preset time delay. If system demand is heavy and the stopping cycle takes place, the first unit is taken off the line, but the second unit remains on the line until the system demand decreases to such a point as to permit the second unit to be shut down.

Automatic Diesel Starters

In certain areas, where reliability of service requires that the pumping plant be available at all times, a diesel engine–driven pump is started automatically in the event of power failure. The automatic diesel starter is actuated by a combination of line voltage failure and low pressure on the system. When both these conditions take place, a circuit is completed to a cycle timer, which initiates engine cranking by connecting the battery to the engine starter. The cranking cycle consists of 10-sec cranking and 5-sec rest periods and may be repeated any desired number of times up to the total length of 5 min. When the diesel engine is started, it is automatically placed at idling speed for 1 min before being thrown over to full speed and coupled to the pump by a centrifugally operated clutch. The diesel is stopped automatically when the pressure on the line reaches a preset value or when the line voltage is restored at the end of the power failure. This disconnects the diesel from the full-speed position and disengages it from the pump. The diesel continues to run for another minute at half speed before being stopped completely. In the event of functional failure of the equipment or if the diesel does not start after the predetermined number of cranking cycles, it is automatically locked out and a white pilot light is energized to indicate that the engine failed to start after having gone through the full

cranking cycle. The automatic controls will lock out the diesel if the engine water temperature becomes excessively high or if the engine oil pressure drops below a safe value, and will also light a red warning light, indicating oil or water failure. Except during special tests the diesel units have always started on the first one or two cranking cycles.

Automatic diesel starter controls are equipped with a four-position selector switch for selection of automatic or manual operation, one "off" position and one "test" position. When in the manual position, all the automatic controls are bypassed and the engine is started by means of a manual-start push button. In the test position, the low-voltage contacts and the low-pressure contacts, which are normally used to start the cranking cycle when the power fails, are bypassed to permit the operator to try out the engine-starting sequence without having to deenergize the pumping plant. The automatic starter controls on the diesel also incorporate a trickle charger for charging the 24-v engine-cranking battery. The charger consists of a dry transformer, selenium rectifier, current-adjusting variac, and an ammeter to indicate the rate of charge.

In the Los Angeles system there is one plant, in a high restricted residential district, which is rather unique in that submersible-type booster pumps are used. Because of the small rotating mass in the submersible-type pump, the units come up to speed almost instantly and as a consequence the plant was faced with a problem of extreme water hammer. In this one plant it was necessary to redesign the control system so that when the pressure switch was actuated to bring a pump on, the pump started against a closed discharge gate, which was then opened very slowly by a hydraulically operated cylinder. As the tank filled, and the desired pressure was created, just the reverse happened. The "stop" mercoid would trip, closing the discharge gate, after which a microswitch installed on the check valve would shut the pumping unit down until the next cycle of operation. This system was further improved by running a selector system so that each unit in the system had the same running time. Thus, in periods of extreme demand, all three units in the plant could operate simultaneously—with never more than one start or stop at the same time, however, to avoid the water hammer problem mentioned previously. The only weakness in this system is the possibility of a power failure. To forestall such an occurrence, a relief valve has been set in the system which will discharge back into the suction side of the units.

Experience With Automatic Plants

Experience within the last 10 years has shown the automatic pumping plants to be most satisfactory. Reliability of these plants is extremely high, and maintenance is kept at a minimum by proper design of components. The automatic selection system used in these pumping plants has been standardized as much as possible so that most installations completed after 1953 are basically identical in design. This permits a great saving in spare parts, and also allows the maintenance man to become thoroughly familiar with the circuit. Another method used to simplify maintenance is the use of plug-in relays throughout the plant. If one of these relays fails, only a few seconds are required to pull it out and plug in a spare unit, putting the circuit

back in operation. The actual performance of these plants appears to be better than that of the manual plants; this may be explained by the ability of the automatic controls to maintain a constant watch over the system and correct any deviation from the desired conditions within a few minutes.

Economic aspects of automatic pumping-plant control reveal a great saving in operating costs against corresponding performance of a manually operated plant, which requires three shifts of operators plus the necessary standby personnel to relieve these operators in case of sickness or vacation. Assuming that the operator's pay is \$2.50–\$3.00 per hour, including the necessary overhead expenses, the daily cost for labor to operate a plant will be about \$60–\$72 a day, or approximately \$22,000 per year. If the control equipment can be capitalized at 10 per cent, it means that some \$220,000 worth of equipment may be installed in a plant to save the expense of the labor. Actually the automatic equipment for a pumping plant costs only \$3,000–\$5,000, but maintenance of the equipment must continue as before. The saving in operating expenses, however, is appreciable. Not all pumping plants, of course, share the operating characteristics or conditions here described; but through the standardization of controls and control systems, it is possible to achieve important economies as well as increased reliability of the system.

At the present stage of development, the automatic pumping plants in the Los Angeles system are highly successful and economically rewarding. They do, however, require careful planning before installation. The Los Angeles system is now looking forward to the time when the telemetering of the functions of a few of the major plants will be effected. Such a system is partially in service already, where reservoir readings, pipeline flows, and pressures are telemetered. Considering the reliability of well designed automatic pumping plants, the cost of telemetering is not justified except in special cases.

Acknowledgment

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Water Hammer Problems at St. Louis

Frank E. Dolson

A paper presented on May 9, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by Frank E. Dolson, Supt. of Dist., St. Louis County Water Co., St. Louis, Mo.

BECAUSE of the present tempo of expansion at St. Louis, it has not been possible to secure complete data on the operating characteristics of the surge control equipment installed on the recently constructed additions to transmission mains. The information presented here should therefore be considered a preliminary report on the methods employed to confine surges to manageable and safe amounts. It will be descriptive rather than theoretical, with very little, if any, contribution to the knowledge of water hammer analysis.

Design Problems

The water hammer problem is very complex. In simple conduits of uniform thickness and diameter, with known elastic properties and length and with no branch lines, there is often a remarkable correlation between computed and measured values. But, in conduits with variable thickness or diameter or in conduits having branch pipes, the analysis of the problem is made much more complex by reflected waves. In pump discharge conduits the problem is still more difficult because of such factors as the rate of discharge valve closure, the rotation effect of pumps, pipeline profile, and the dampening effect of the distribution system. All of these factors may affect the shape, amplitude, or period of the fundamental or secondary waves, and it is not always possible to compute or predict the influence of these factors, either individually or collectively. The designer is confronted with these problems, however, and, before the pipeline is placed in operation, he must design and specify equipment which is capable of reducing surges to safe limits. As is often the case, these circumstances result in design that is on the conservative side. The designer provides more surge valves than are actually needed. Vacuum and air relief valves of either the quick-opening and quick-closing or quick-opening and slow-closing type are installed along the pipeline at every location where it appears that they may be needed. In addition, other connections are often provided so that the original equipment may be supplemented at a later date. Transmission pipeline failures are not only expensive to repair but often result in serious water service curtailment for an extended period of time. In view of this, one can hardly blame the designer for taking a conservative approach to the surge problem and providing more than a bare minimum of equipment.

St. Louis Equipment

Surge control equipment for the transmission mains of the St. Louis County Water Company has been designed on this basis. Surge relief valves, set to open on the depressed-pressure wave or down surge, are installed on the transmission pipe near

the discharge pumps. Vacuum and air relief valves of the slow-closing type are installed at critical points along the profile of the pipeline. At other points along the profile, smaller vacuum and air relief valves of the quick-closing type may be installed if circumstances indicate a probable need. If two relief valves are required to control the surge at maximum design flow rates, three valves are provided so that two will be available even though one fails to function. In the same extremely conservation manner, vacuum and air relief valves are installed in pairs.

Three transmission main systems are now being used to provide water to the 210-sq mile service area. In June of 1956 a fourth transmission main was placed in operation. Only the first two will be discussed here, however, because they represent typical surge control problems (Fig. 1).

High-Service-Low System

The first system, known as the Central Plant high-service-low system, consists of three parallel cast-iron pipelines, 20, 24, and 36 in. in diameter, which connect to the pumping station on one end and to an 11-mil gal reservoir on the other end, where all the water is repumped. The lines are 39,100 ft in length and traverse a profile which rises 180 ft within a short distance from the plant, then drops off into a valley from which it gradually rises to the reservoir elevation.

The input into the lines is by five electrically driven centrifugal pumps. Two of these units have a capacity of 18 mgd each. The others, which are of the outdoor vertical type, are smaller.

Midway between the main pumping station and the reservoir, a booster pump equipped with three electrically driven centrifugal pumps and having a total capacity of 48 mgd, has been installed. Suction to the pumps is from all three pipelines, but only the 24-in. and 36-in. pipelines are used to convey the water from the booster pumping station to the reservoir. The 20-in. pipe at this point is converted to distribution pressure by an additional pump which takes suction from the discharge of the other pumps.

Without booster pumping, the system has a capacity of 36 mgd. With booster pumping, its maximum capacity is 48 mgd. The system is flexible and almost any flow rate can be obtained by selecting the proper combination of pumps.

Water hammer experience with this system dates back to 1932 when the steam-driven pumps were replaced by electrically driven ones; the steam-driven pumps were then relegated to standby service. The difficulties encountered and the corrective measures employed are described by Lischer (*1*).

A plan and profile of the three pipelines, as they existed after the electrically driven pumps were first installed, are shown in Fig. 2. As the system was arranged after the electrically driven pumps were installed, there were several trouble spots, with possibilities of water column separation at two points. As a result of the original study, three 6-in. air valves, one connected to each of the three pipelines, had been installed at the highest point along the profile nearest to the pumping station.

A restudy of the system at the time the booster pump was constructed indicated that the original equipment had sufficient capacity for flows up to 45 mgd, if some means were provided to relieve the effect of any surge on the reservoir side of the booster station. Such relief might be accomplished either back through the pumps or through some bypass arrangement to

the surge relief equipment installed near the main plant. The booster pumping station was designed on the basis of a 45-mgd capacity. A spare pumping unit was also provided for reliability purposes. By the use of a third pumping unit the capacity of the system could be increased to 48 mgd, but only at the risk of inadequate water hammer protection.

In the summer of 1952, when the system usage exceeded available capacity, the spare unit at the booster pumping station was used to increase the flow to 48 mgd. During one of these periods a surge of unusual magnitude occurred, not only breaking the pipeline at one potential trouble spot, but also causing considerable damage to the pipes within the booster station. Fortunately, the pump casings were not damaged, although all the pumps had to be realigned because of distortion occurring in the discharge header. It should also be mentioned that damage to the other equipment and building was minimized by the fact that all piping within the building was made of fabricated steel joined together by flexible couplings, with steel tie rods provided to take care of unbalanced thrusts.

A review of the events preceding and at the time of the failure indicated the following:

1. While putting the booster pumps into service to increase the rate of flow from 36 mgd to 48 mgd an interruption occurred, causing surge of unknown amplitude.

2. The flow interruption was caused either by an error on the part of the operating personnel or by equipment failure; it was not caused by a power outage.

3. It was not possible to reconstruct the sequence of events immediately prior to the failure.

4. The surge relief equipment installed between the booster pump and the main pumping station protected that portion of the pipeline.

5. The water column separated at the highest point on the profile (4,000 ft east of the booster pumps), and upon rejoining, without the benefit of vacuum and air relief, caused a secondary wave which evidently superimposed on the primary wave to cause destructive pressure.

6. Vacuum and air relief were needed on each of the pipelines at the point of failure at flows exceeding 45 mgd, and such relief was probably advisable for smaller flows in case a similar sequence of events should be repeated.

After further study, it was concluded that the system could be protected from a water hammer surge resulting from the interruption of a 55-mgd flow rate if the installed relief equipment was supplemented by:

1. Individual 4-in. vacuum and air relief valves installed on each of the pipelines at the highest point along the profile.

2. An additional 6-in. angle needle relief valve installed near the main pumping station and controlled so as to open at a predetermined point on the down surge, thus serving as a reserve for either of the other two valves

3. Controls to insure that the discharge valves of the booster pumps remained open during any surge period resulting from an interruption of flow so as to make the pipelines hydraulically continuous between the suction and discharge sides of the booster pumps.

This completed installation of surge control equipment, consisting of three surge relief valves installed near the pumping station and two sets of vacuum and air relief valves installed at

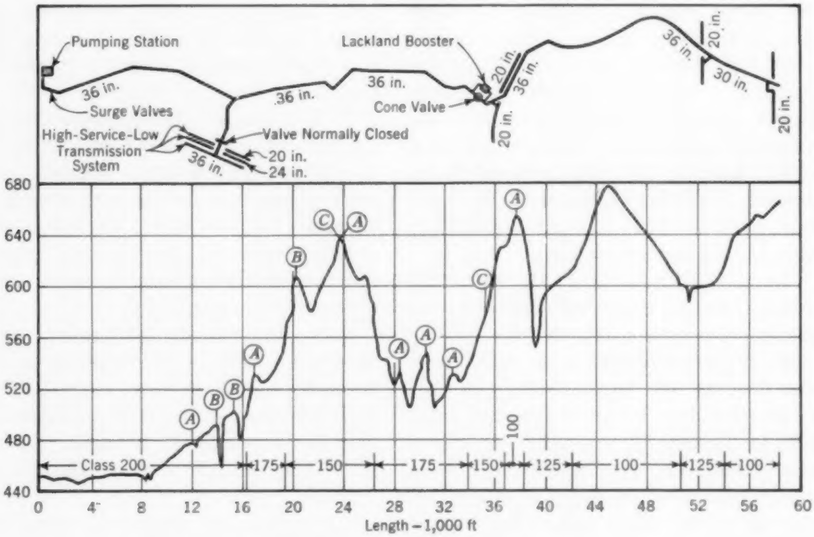

Fig. 1. High-Service-High System Plan and Profile

The circled letters in the profile indicate the following: A—*two 2-in. quick-opening, quick-closing vacuum and air relief valves;* B—*two 4-in. quick-opening, quick-closing vacuum and air relief valves; and* C—*two 4-in. quick-opening, slow-closing vacuum and air relief valves.*

the two critical high points along the profile of the pipeline, has proved effective in controlling and confining surges to acceptable limits at flow rates of 48–54 mgd.

High-Service-High System

The second transmission main, known as the High-Service-High System, presented a more challenging problem of water hammer design than did the original system. More unknown factors were involved, the influences of which were difficult to evaluate. The profile along the route of the pipeline was rugged, necessitating frequent changes in the class of pipe used to obtain minimum installation cost and consistency in safety factor. A large-capacity booster pump further complicated the problem. In addition, this pipeline discharged directly into the distribution system, which introduced the customer factor. Not only was the water hammer length of the pipeline (or point of wave reflection), unknown, but the velocity of the wave for the type of pipe used was not available from the manufacturer, nor had it been measured by other users.

A diagrammatic plan and actual profile of the pipeline are shown in Fig. 1. The pipeline, consisting of 52,000 ft of 36-in. and 6,000 ft of 30-in. prestressed steel cylinder concrete pipe, connects the new 36-mgd central plant addition directly to the distribution system. Major connections to the distribution system are provided at points which are 37,000, 52,000, and 58,000 ft distant from the main pumping station, respectively.

It will be noted that this transmission main is interconnected to the orig-

inal transmission main system by a 30-in. bypass pipeline. Normally, the systems are isolated from one another by a closed valve, but under emergency conditions, by revalving, great flexibility can be obtained.

At the main pumping station, the pipeline is connected to three two-stage electrically driven centrifugal pumps having capacities of 14, 16, and 22 mgd at heads in the 150–200 psi range. As shown in Fig. 3, a 36-mgd booster pumping station has been installed on the transmission main approximately 2,000 ft before the first major system connection. The capacity of the pipeline without booster pumping is 27 mgd, and 36 mgd with booster pumps.

The booster station merits special mention not only because it is remotely controlled from the control room at the central plant pumping station by supervisory control equipment, but because it introduced another factor into the water hammer problem which required special consideration.

Four 250-hp diesel-driven centrifugal pumps are installed in this station. Water is pumped around a closed 24-in. cone valve which can be opened or closed as needed by the remote controls provided. To protect the pipeline from overpressure, in the event that one or more of the diesel units fail when being operated, an interlock which will cause a selected pumping unit at the central plant to trip is provided. In this case, the cone valve opens slowly and the booster pump discharge valves close slowly. Should one or more pumps at the central plant trip because of a power outage or for any other reason, pumps at the Lackland station, unless opened by a remote manual control, continue to run with the cone valve closed. In all but unusual cirumstances, the surge valves installed on this line would open, as well as one or more of the vacuum and air relief valves. But, unless the cone valve at the Lackland Station were open, the static pressure at the surge valves was insufficient to cause reclosing. Thus another factor requiring consideration was introduced.

The design of the surge control equipment was predicated upon the following conditions and requirements:

1. The design of the pipeline was to be on the basis of a 36-mgd rate of flow.

2. Pumpage through the pipeline would be directly into the distribution system at distribution pressure.

3. To obtain the design rate of flow, booster pumping would be required; this introduced the problem of reclosing the surge valves after operation.

4. The design of the protective surge relief equipment was to be based on confining surge pressures to limits only slightly in excess of the pipe class and in no case to exceed the water hammer allowance provided for in the design of the pipe.

5. To prevent secondary surge waves from occurring as a result of water column separation at high points along the profile, sufficient vacuum and air relief valves were provided at vulnerable points.

6. Distribution disturbances during all phases of operation, including the operation of the surge control equipment, was to be held to a minimum.

Final design of the surge control equipment provided for the following:

1. Three 6-in. angle type relief valves were to be installed on the pipe near the main pumping plant. Two of these valves were needed to protect the lines against water hammer surges at the maximum design rate; the third unit was installed for safety reasons to insure that at least two relief units

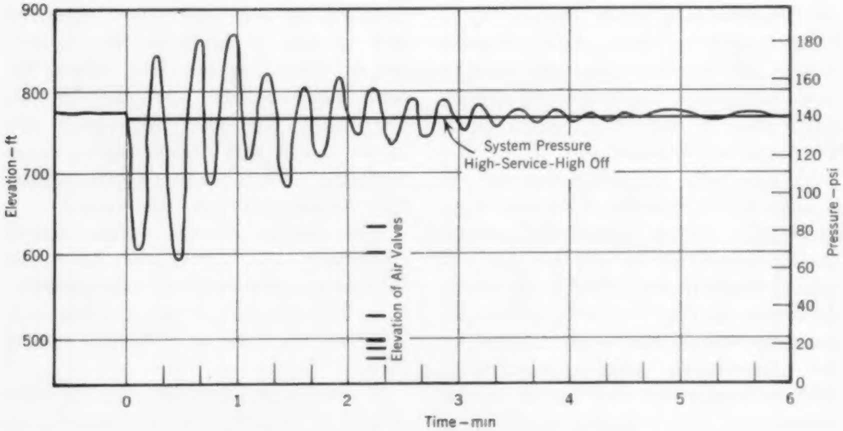

Fig. 2. Pressure Wave in 36-in. Discharge Pipe

To stimulate a power failure, the breaker was tripped at 0 sec. This interrupted a flow of 6 mgd having a velocity of approximately 1.31 fps. Air valves were in service and surge valves were not. Elevations given at the left are from sea level datum. The elevation of the surge recorder making the measurements was 446 ft. The straight solid line is for system pressure off. During the test, the highest air valves did not open.

would be available even though one failed to operate. These valves were set to open whenever the depressed-surge pressure fell to 90 psi. Reclosing was timed so as to prevent objectionable penstock type of surges.

The valves are equipped with high pressure controls set to open them at 215 psi, which is 15 psi more than the highest operating pressure. This feature not only protects the pipeline from excess pressure resulting from mistakes in operation, but, more important, affords protection against faulty operation of the interlock between the central plant and the booster station.

Because of insufficient static head to reclose the surge valves after opening when the cone and pump discharge valves at the booster station were closed, it was necessary to provide water at supplementary pressure as well as additional pilot valves. The operator, by the use of a push button, can reclose the valve from the control room. By indicator lights, he can determine whether the controls are reset, and thus protect against subsequent surges.

2. Vacuum and air relief valves were installed at the high point along the pipeline at more frequent intervals than usual because of factors associated with the cone valve at the booster pump.

3. Remote manual control of the cone valve was provided for distribution system protection.

Figure 2 shows a replotting of a pressure wave obtained on a high-speed pressure recorder directly connected to the pipeline near the main station discharge valves. The purpose of the test was to determine the critical time of the pipeline and the point of wave reflection without any of the surge control equipment in operation. It was also thought that relief afforded by the distribution system would be apparent on the chart. After the rate of flow had been reduced to a safe

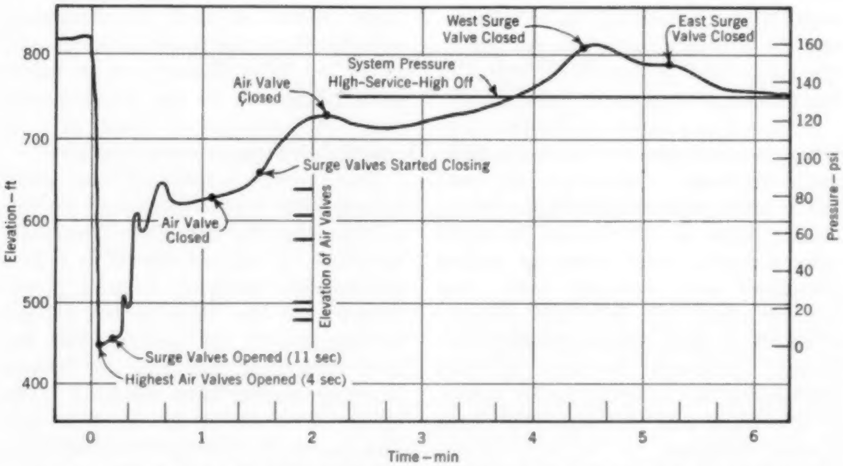

Fig. 3. Pressure Wave in 36-in. Discharge Pipe With Surge Valves in Operation

To simulate a power failure, the breaker was tripped at 0 sec. This interrupted a flow of 17 mgd having a velocity of 3.72 fps. Both surge valves and air valves were in service. The straight solid line is for system pressure off.

amount by throttling one of the discharge valves, the breaker was tripped to simulate a power failure. The surge relief valves were not in service at the time of the test and the air valves, although operative, did not function. The pressure wave shown in Fig. 2 is the surge effect caused by interrupting a flow of 1.31 fps in this pipeline.

The critical time (time interval between successive crossings of the static line by the pressure wave) was 10–12 sec. Assuming that the velocity of the pressure wave is 3,400 fps, which is the approximate mean velocity reported for pipe of the same design and having similar elastic properties, the point of wave reflection is about 17,000 ft from the pumping station. Wave reflection from this point may be caused by either the closed valve on the 30-in. bypass between the two transmission systems or by the different elastic properties of the pipe resulting from a change in pipe class.

It was a surprise to find the point of reflection only 17,000 ft distant from the plant, because it had been anticipated that the reflection point would be either at the first system connection to the pipeline, some 37,000 ft away, or at some point beyond. The irregularity of the peaks in the wave shown in Fig. 2 may indicate that there is another wave of different critical time present. This could be the fundamental wave for the entire system but the wave form justified only speculation and not a definite conclusion. It would be interesting to observe the wave period obtained by interrupting a much higher velocity, but the possibility of destructive pressure prevents this.

In an effort to determine the adequacy of the surge relief equipment installed on this pipeline, a power failure condition was simulated, interrupting the flow having a velocity of 3.72 fps. Again, a high-speed pressure recorder was attached to the pipeline

near the pump discharge valve to measure the pressure variations caused by the surge and to record the effect of the protective equipment.

Figure 3 is a replot of the pressure-time relationship measured on the high-speed recorder. Observers, stationed at the surge valve and the highest slow-closing type of air valves, in radio communication with pumping station personnel and equipped with stop watches, noted the time and pressure at which the protective equipment functioned. Although observers were not stationed at the intervening air valves, it is evident that the depressed wave opened these too.

The velocity of the flow interrupted in this test resulted in a depressed wave which caused negative pressure at the surge valves located a short distance from, and only slightly higher than, the pumps. Complete protection of the pipeline from the surge was obtained by the relief equipment. This test further indicates that the pipeline is fully protected against surges resulting from an interruption of the designed 36-mgd rate of flow.

It will be noted in Fig. 5 that the highest air valves, which are 24,000 ft distant from the pumping station, opened in 4 sec, while the surge valves which are only 1,100 ft distant, opened in 11 sec. The sequence of events was questioned, but a subsequent test run under identical conditions gave the same results.

The inertia characteristics of the surge valves may account for the time delay in their opening, but it is difficult to understand the reason why air valves open in 4 sec. The depressed wave could not have reached these valves in less than 7 sec if the speed of the wave front were approximately 3,400 fps. It appears that these valves opened simultaneously with the first, and possibly the second, set of the quick-closing type of valves installed nearest to the pumping station. No attempt is made here to explain this apparent enigma.

One failure attributable to water hammer has been experienced on this pipeline and, like the failure previously described, it was not caused by a flow interruption resulting from a power failure. At the time of the failure, construction of the pipeline had not been completed, nor had the booster pumping station been installed. The failure occurred at an overbend and was caused by a surge resulting from revalving the pipeline so that a portion of it could be placed in operation through the bypass to the original transmission mains. Although the surge which caused the failure resulted from a procedural error, it served a useful purpose in that it revealed the need for considerably more air relief than originally installed.

Conclusions

While conclusions are not necessary in a descriptive article such as this, it would be an omission not to state that direct pumping, by electrically driven centrifugal pumps to a distribution system and through a long transmission pipeline traversing unfavorable topography, presents many design problems not encountered in other transmission pipelines where discharge, for example, may be to an open reservoir. It should be stated that little is known about surge relief afforded by the distribution system.

Reference

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Problems of Penalty Rates and Control Facilities

Vance C. Lischer

A paper presented on May 9, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by Vance C. Lischer, Partner, Horner & Shifrin, Cons. Engrs., St. Louis, Mo.

THE expansion of large metropolitan areas has created unusual problems of water supply With improved living standards and the related greater use of the automobile for all family transportation, as is often the case, living has become more spacious To meet this trend, water distribution systems have been extended over great distances creating new problems of pumping and supply.

There are many instances where small communities or subdivisions have banded together for the purpose of providing water supply and other municipal services through the formation of water districts or other types of political subdivisions and have contracted for water service from a nearby metropolis. In most cases the loads thus connected are purely residential and, as such, have poor load factors and high peak demands.

Large metropolitan centers have often engulfed selfcontained communities which formerly provided complete municipal services including water supply. This expansion has sometimes brought the water distribution system of the parent metropolis to the boundaries of the smaller community. The possibility of purchasing water from a larger and more economically produced supply presents an opportunity for economical expansion of the smaller community's water system.

Such rendering of water service to the suburban population in wholesale quantities for which adequate rates must be established has brought about a realization that commodity charges alone do not properly compensate the supplier for the plant and transmission mains required. Progressive suppliers of wholesale water, therefore, are invoking rates with demand provisions, and, as a result, the user must develop a means of control if costs for purchased water are to be kept at a minimum.

The purpose of this discussion is [1] to outline the problems of controlling the supply of water in a system where demand considerations of one form or another are included in the wholesale rate, [2] to suggest methods of control, and [3] to describe an installation where such means of control are provided.

The situations under consideration here are assumed to be such that manual, attended control cannot be economically justified and that remotely controlled or semi- and fully automatic controlled installations must necessarily be used.

It is not intended to discuss the merits or equity of possible demand or penalty rates. Because rates are subject to change, the design of a facility for operating under such rates should necessarily be flexible.

System Characteristics

In order for the rate of supply to be varied, as it must be if it is to be controlled to meet a penalty or demand rate, there must be sufficient equalizing storage in the system to permit at least a uniform inflow rate on the maximum day, and more if the penalty rate calls for reducing the rate of water input during certain hours of the maximum day. Generally, equalizing storage in the amount of 15–25 per cent of the maximum daily usage is necessary if a uniform rate of input is employed on the maximum day. If penalty rates are in use, a larger amount of equalizing storage may be necessary.

When a supply is purchased from a system which has its own peak demand problems, there is opportunity, through the use of storage in the supplied system, to obtain a lower rate for purchased water by submitting to reduced rates of input during the peak hours of the supplying system. With the use of repumped ground storage, allocating a large amount of storage for equalizing purposes is economical and can be used to encourage application of lower rates for service.

Need for Pumping

The need for pumping of the purchased supply is independent of the means of control, although, where pumping is necessary and system characteristics are sufficiently constant so that the pumping rate is relatively uniform, there is opportunity to use time control of operation under certain types of penalty rates. This will be discussed in detail later.

Whether or not pumping is required is dependent entirely on the relative gradients of the two systems. Because there are unavoidable losses due to metering and control, some additional energy can be expected to be needed in most cases. The suppliers' meters probably will be of the high-accuracy, displacement or turbine type, and losses of 5–15 psi can be expected. Adequate rate indication for operating a control valve and the control valve itself may call for 1–4 psi of additional loss. It is likely, therefore, that some pumping will be needed.

If the ground storage facility is situated so that the supply can be admitted into it for repumping during peak hours or peak days, it may be possible to design a system in which one pumping installation at the ground storage tank site will suffice. During other periods the normal gradient of the supply may be adequate without repumping.

Metering Problems

Up to the present time there have been no well developed demand meters designed for the normal wet and unheated meter vault customarily used for customer metering. In any case, a conventional meter vault is not a suitable place for the recording chart instruments currently available.

A basic problem still exists in rate-determining appurtenances currently available for volumetric meters: if such meters are to indicate an instantaneous rate, it is necessary for the movement of a rotative shaft to be converted into an instantaneous rate of speed. The mechanism which accomplishes this is not as rugged as the meter itself, and generally it is not suited to installation in a conventional meter vault. As an alternative method to the direct recording of instantaneous rate, it is possible to use a recorder which develops a mass curve. Instantaneous rate can be determined by calculating or measuring the slope, but this is laborious and highly unsatisfactory.

The measurement and recording of average rates for fixed intervals, such as with the 15-min demand instruments used for the sale of electric power to large power users, have not been developed for general use in the water-metering field. The development of devices which could be installed in the unfavorable environment of an underground meter vault would not seem to be the best answer for the demand-metering problem encountered in serving water wholesale to municipalities or water districts. The solution to this problem of determining instantaneous rate would better be integrated into the means of control provided by the customer. That, at least, is the simplest answer, pending the development of a rugged control device with flexible rate variation, using the conventional volumetric meter as the sensing device.

The integration of the two requirements—that of recording or determining rate of flow with respect to time of day by the wholesaler, and that of controlling rate of flow by the purchaser—would involve common use of the rate meter and recording device. Thus, the wholesaler would use the customary volumetric meters only for the purpose of measuring total volume used. The control of rate need not be exact, and inaccuracies even up to 5 per cent may not be too serious. From the standpoint of administering the account, it would be better if there were only one rate-of-flow determining device, jointly maintained and checked by both parties. This would avoid conflicts which might occur because of disagreement between separately maintained and operated rate-of-flow devices.

For reasonably accurate rate measurement and control, the use of some form of differential type meter appears to be the only recourse. Since the process of control involves throttling for most operating conditions, the use of an orifice meter can be justified. Such a meter offers great flexibility in that the orifice size can readily be changed and the range of control can therefore be broadened at low cost.

Consideration must also be given to the characteristics of the control valve, but these are generally not as restrictive as the range of a single orifice. Figure 1 shows an arrangement of control limited to a possible range of 10 to 1 if a single orifice is used and head losses are to be kept reasonable. By changing the orifice plate, this range could be extended to the limitation of the range of the control valve, which might be as high as 20 to 1 with a butterfly valve, and higher for characterized globe types. With a venturi meter being used, the range of the system could be extended to possibly 20 to 1 with a single chart and with no changes being required as the demand on the facility increases.

The range of control required of an installation will depend on the characteristics of the system it serves. Normally, the restrictions on maximum rate of input will occur during summer months. During the other months of the year there probably will be no penalty charges. If the supply, subject to demand or penalty rates, is the only source and if equalizing storage is available, the control might be designed for a 3 to 1 range with a 2 to 1 growth range superimposed, making the total range 6 to 1. This is not a troublesome range for which to design.

If the community has another supply and the purchased supply, subject to demand and penalty rates, is auxiliary, the range of control required can be extreme. This is true for the installation described in this article, which is for the city of Kirkwood, Mo.,

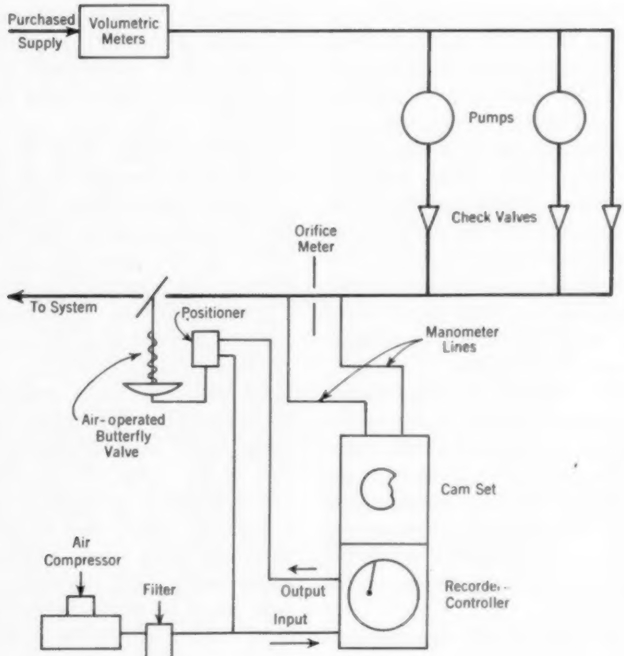

Fig. 1. Simple Control System

a suburb of St. Louis. Kirkwood has a plant on the Meramec River and a connection for a portion of its supply from the system of the St. Louis County Water Company. The type of rate offered for service gives opportunity to benefit from the limited water use which might occur in a wet summer. In such a situation, little supplemental water would be needed because the plant can supply all normal demands. In a dry summer, a high rate of use from the auxiliary supply can be experienced with attendant high monthly charges for 12 subsequent months. When this situation is superimposed on growth potential, which is here expected to be met by the auxiliary supply, a range of control of 30 or 40 to 1 is indicated.

Because the required range was beyond the practical limits of a single orifice or venturi and a single-size control valve, a dual-range installation was conceived. A schematic diagram of this installation is shown in Fig. 2. It has a single differential meter and controller with two orifice plates and air-operated butterfly valves. An arrangement using two orifice plates interchangeably in the same line was not considered desirable because dual control valves were required and because the inconvenience of changing orifices was to be avoided.

Methods of Control

The problem of control can be divided into two separate categories. The first relates to integrating the facility into the water requirements of the user, and the second relates to the need for limiting the rate of input in

accordance with the stipulations of the demand or penalty restrictions.

With regard to the first category, the means of control must satisfy the system characteristics. If the purchased supply is the sole supply to the community, the input must be controlled in conjunction with the equalizing storage facilities of the system to keep the community adequately supplied at all times. This might mean level control of the pumping units or throttling control from the storage facility. In this regard, the control can be made typical of those customarily used for automatic control of a supply.

If, on the other hand, the purchased supply subject to demand or penalty rates is an auxiliary supply, a criterion of operation must be established, and this will probably be dependent on the individual circumstances. An automatic or semiautomatic control can be worked out using conventional components to meet almost any difficulty in coordinating the operation of the auxiliary supply with the principal supply. If, as is the case with the Kirkwood installation, the principal supply is an attended plant where full information is available through telemetering circuits, the starting and stopping of pumping units at the purchased supply can be by remote control. This method is simple and reliable.

With regard to rate-of-flow control, there are many satisfactory systems. A demand or penalty rate will require that rate of flow be controlled with respect to time, and there will probably be seasonal and hourly stipulations.

It is likely that only the summer season will involve flow limitations. In an area with climatic conditions typical of most of the United States it is probable that all months except June–September will be without restrictions.

During the summer months, the rate will probably have a penalty for excessive rates of input during the normal peak sprinkling hours. The rate may possibly encourage depressing the rate of input during these hours to benefit the supplying system; lower rates for water service will probably result. The development of such opportunity is inexpensive when a repump ground storage facility is available.

In the case of the Kirkwood situation, a rate patterned after the large manufacturer rate of the St. Louis County Water Company was offered. The special rate, with the penalty stipulations that resulted, was designed to make the input characteristics of the Kirkwood connection perform in a similar way to that of a manufacturer. The stipulation was that the rate of input during the peak sprinkling period be dropped on a maximum day. The other facilities of the system—a repump storage facility, a standpipe, an elevated tank, and the separate plant—gave opportunity to take advantage of this rate, which is $90 for the first 60,000 cu ft or less per month, as registered by meter; for quantities over 60,000 cu ft per month, the rate is 10½ cents per 100 cu ft.

The rate carries a stipulation concerning minimum charge, which is that the maximum daily rate of usage during June–September shall not exceed twice the average daily rate for the month, and the maximum hourly rate of usage during the hours of 4 PM–10 PM shall not exceed the average hourly rate during the monthly billing period. If these ratios are exceeded, however, the minimum charge during these months shall be based on the greater amount computed either as fifteen times the maximum daily usage occurring in the monthly billing period, expressed in cubic feet, or 30 times the maximum

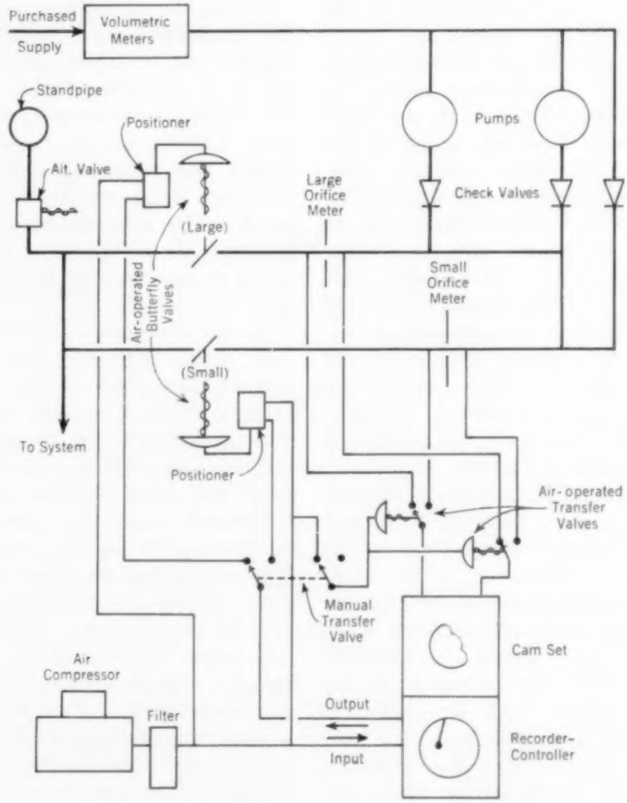

Fig. 2. Schematic Diagram for Swan Station

hourly rate occurring between the hours of 4 PM and 10 PM during the monthly billing period, expressed in cubic feet per day. The minimum charge in any month shall be 60 per cent of the maximum charge calculated from the actual usage and rates of flow during any of the months of June–September in the 12-month period preceding the month for which the bill is to be rendered, but not less than $90.

The rate and time settings of the control must be determined by someone in the consumer system. The settings should be determined on the basis of progressive changes in the water demand during the critical season establishing the minimum bill for the subsequent period, as may be indicated in the rate. This operation requires that the person responsible visit the station and make the necessary setting for limiting the inflow which is possible through the remote or automatic control operation of the facility. Normally, the changes in settings would begin at the onset of summer and would continue from week to week, either upward or downward, to assure, first, that the minimum amount of water being established by rate stipulations is being used, and, second, that any increased settling is in conformance with the need for water and,

third, that rates of inflow during the penalty hours are not adversely affecting the cost of water.

There are several possible methods of controlling rate, and a few are described as follows:

1. A standard rate-of-flow controller, preferably air operated and with a time switch to change the control point at predescribed times during a 24-hr period, can be used.

2. A standard rate-of-flow controller, preferably air operated, with a 24-hr cam for giving maximum flexibility in variations of flow and time components, can be used. This is the method of control used in the Kirkwood installation, and is considered preferable to the method described above.

3. Two separate controllers, possibly of the filter rate control type, and with a time switch to transfer from one to the other are used. This method is limited to the establishment of two flow rates for automatic control unless a resetting is made within the 24-hr period, and would seem to be costly and does not have the desired flexibility.

4. If the rate stipulations are concerned only with average rates of flow during the noncritical hours of the day and only with instantaneous rates during the peak hours, a combination of time control and rate control can be used. Since a pump used for admitting water will have an output which will probably not vary more than 10–15 per cent, the use of a timer to limit the hours of operation during the noncritical period of the day would provide satisfactory control of average daily rate except for the usage during the peak hours. A separate pump, throttled to the desired peak rate, could be used to limit instantaneous rate during the peak hours. With suitable time switches, timers, and interlock, a remote operator could be prevented from admitting more than a predescribed maximum rate during the peak hours subject to the 10–15 per cent variation in pump output. This would seem to be sufficiently close for most operation.

5. A desirable method of control would involve use of a scheme for which standardized and well developed components are not now available. This method would use the volumetric meter of the supplier as the basis of the control scheme and would not require conversion of the totalizer action into rate indication to initiate control.

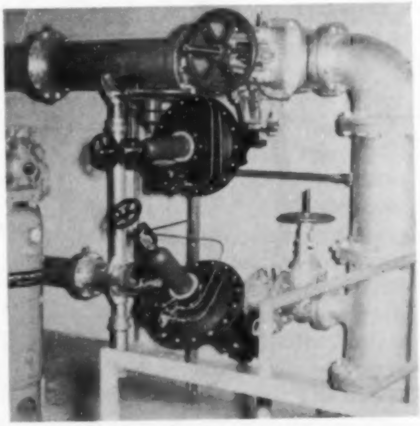

Fig. 3. Butterfly Control Valves

The top control valve is an 8-in. rubber-seated air-operated butterfly valve, and the bottom one is a 4-in. butterfly valve.

This means of control would involve the pacing of the rotation of the register in accordance with a pace-setting rotating device whose rate of rotation can be preset and controlled for the desired 24-hr demand schedule. The control would be effected by a differential device which would control the flow regulation valve to make the two rates of rotation the same at all times. It is conceivable that an air controller to operate a butterfly valve can be developed so that the accuracy of con-

trol could be maintained within a minute fraction of one revolution of the meter spindle. This means of control would be much more accurate than any type of control which can be developed for orifice or venturi metering. It would have advantage in that no additional metering would be necessary beyond the volumetric metering provided by the water supplier.

standpipe and the schematic piping layout is shown in Fig. 2.

The station contains four electrically driven pumping units, two at 400 gpm, one at 800 gpm, and one at 1,000 gpm. A dual gasoline-engine drive on the largest unit permits possible delivery of 1,500 gpm. The gasoline engine is entirely manual in its operation.

The starting and stopping of pumps

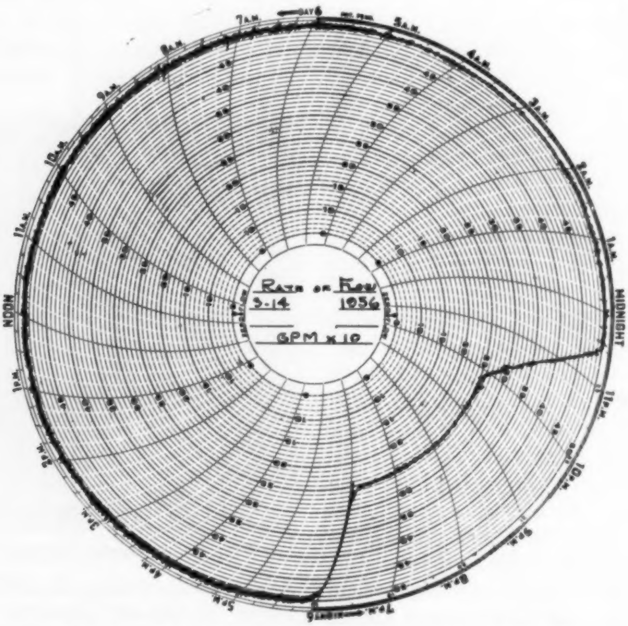

Fig. 4. Flow Chart Showing Effect of Air-operated Flow Controller

Kirkwood Installation

The Swan Pumping Station of Kirkwood, Mo., is the point of input of the auxiliary supply purchased from the St. Louis County Water Company to supplement the supply from the municipally owned plant which has a capacity of about 4 mgd. The facilities at the Swan Station have capacity for admitting about 2.5 mgd. The plant is located on the site of a 350,000-gal

is remotely controlled over telephone circuits from the treatment plant about 5 miles away, where operators are on duty 24 hr a day. Information on the system is telemetered to the plant, reporting the level of the standpipe at the Swan Station, the level of the ground storage tank in the system, and the level of the elevated tank in the system. In addition, the plant has shutoff control of the pumps at the

ground storage installation which are started by a drop in pressure, and the system pressure can be determined at any time by telephone at police headquarters in the heart of the distribution system.

The facility is housed in a ground floor pump station building at the highest point in the city, designed to harmonize with the residential area where it is located. The standpipe at the location is about 50 years old and is somewhat out of harmony.

The facility houses the meters of the St. Louis County Water Company, the altitude valve for the standpipe, the pumps and motors, control air compressor, and the electrical and control facilities. The building has a large ventilating fan thermostatically controlled to ventilate the building at temperatures in excess of 90°F to dissipate the heat from the motors. All input air (except when doors or windows may be opened) is filtered. The station is heated electrically with a 10-kw heater designed to maintain 40°F, thermostatically controlled, during the winter. A dehumidifier with timer control is provided to minimize condensation and moisture problems. An overhead traveling crane of 1-ton capacity is available for handling equipment in the station.

A diagram of the control is shown in Fig. 2. The heart of the control is a cam-set, air-operated flow controller with mercury manometer using orifice meters as the primary device. The control is designed to operate over a range of about 40 to 1 and two orifices are therefore used. Each orifice has a separate control valve. The large orifice is in a 10-in. line and has an 8-in. rubber-seated, air-operated butterfly valve for control and the small orifice is in a 6-in. line and uses a 4-in. butterfly valve. The large orifice covers a range of 200–2,000 gpm, and the small orifice a range of 50–500 gpm. These control valves are shown in Fig. 3.

The change from the large range to the small range and back is made simply by a single transfer switch, as indicated in Fig. 2. The chart and cam must be changed at the same time. The transfer manipulation involves transferring the orifice connections and the air output of the control instrument from one valve to the other. The valves are rubber seated and tight closing, operating on loss of air pressure. The transfer arrangement bleeds the air supply to the valve on the orifice line not to be used, thus closing it. The feature of closing on loss of air pressure makes the facility "fail-safe" in case of air failure, as the valves will close to assure that predescribed rates cannot be exceeded. The air controller is a typical standard air-operated flow controller and is equipped with proportional band adjustment and automatic reset. The valves are ordinary rubber-seated butterfly valves with standard diaphragm air operators equipped with positioners. The valves can be manually controlled by air pressure from the controller or by means of a jack screw at the valve. The operating air pressure is 17 psi and the supply is from a ½-hp compressor with tank.

The cam-set feature of the controller consists of a separate case instrument with connecting link to operate the control-setting pointer and mechanism in the flow control instrument. The cam rotates once in 24 hr, just as does the chart of the recorder. The cam is cut by the user with a metal shears to any shape desired and the aluminum blanks furnished by the instrument manufacturers are calibrated and are exact replicas of the paper charts. The

very nature of the mechanism of the follower and cam arrangement makes it impractical to raise the setting instantaneously, and an inclined slope on the cam, taking from 5 min for small changes to 90 min for half of full-scale changes, is required. This seems to be the only serious disadvantage of this type of control. Drops in rate can be almost instantaneous, limited only by the speed of response of the control. An actual flow chart shown in Fig. 4 illustrates the effectiveness of the cam in controlling the rate.

The control instruments are mounted on a panel adjoining the power control center serving all of the 440-v electrical equipment. The instrument panel contains a clock, the cam-set air-operated controller-recorder, an air pressure gage, the transmitter for tank level, and recording pressure gages on the supply and supplied systems.

The cost of providing the control as installed in the Swan Station is not believed to be excessive. The pumping station was built in 1955 at a contract cost, exclusive of engineering, of $47,-000. The approximate cost of the control elements included in this project, exclusive of installation costs and the cost of the panel, were as follows:

Cam-set recorder-controller	$ 900
Orifice plates	75
Transfer switch and auxiliaries	125
4-in control valve, complete	600
10-in. control valve, complete	700
Air compressor and filter	250
Total	$2,650

Summary and Conclusions

Modern living has increased the areas served by large water suppliers and has brought with it the wholesaling of water to suburban municipalities and water districts. Demand or penalty rates have thus become more common.

In order to purchase water at the minimum cost under such rates, control facilities are necessary and, in addition, the purchaser's system must contain storage facilities in order that rates of input can be controlled to conform to the demand or penalty condition of the rate.

Rugged and dependable demand or rate-indicating appurtenances to the conventional, high-accuracy, volumetric type of meter used for large customer metering are not available, retarding the application of demand rates.

The problems of measuring, recording, and controlling rate with respect to time can be solved to mutual advantage by the water purveyor and user by means of conventional and dependable mass-produced control components.

The use of the cam-set air-operated controller-recorder, as it is available from a number of industrial instrument manufacturers for this service, using an orifice or venturi primary device and butterfly valves for control, is a satisfactory and economical solution of the problem.

There is opportunity to develop a high-accuracy control using the conventional volumetric meter spindle rotation without converting such rotation into a rate. This would eliminate the need for an additional differential meter to develop rate or to convert the spindle rotation into instantaneous rate indication. To accomplish control, a controllable pacing device can pace the meter spindle rotation and the deviation can regulate the control valve. Thus, accuracy could be controlled within a fraction of a revolution of the spindle. To the author's knowledge, devices for accomplishing this method of control are not available in standardized production components adaptable to this problem of demand and penalty rates.

Comparison Studies of Diatomite and Sand Filtration

George R. Bell

A paper presented on May 7, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by George R. Bell, Sr. Research Chemist, Celite Application Research Section, Johns-Manville Research Center, Manville, N.J.

PROMPTED by a lack of specific information on the applicability, cost, and effectiveness of the diatomite filtration process for water purification, the studies described in this article have been conducted over a period of 6–7 years. The tests themselves were first begun with pilot plant equipment having 1 sq ft of filtering surface, and have since resulted in the installation of a 450-sq ft filter unit. This larger unit has now been operated for a sufficiently long period to make an analysis of its performance both useful and reliable.

Early impetus was given to this project by the adoption of diatomaceous-earth filters for military use, as reported by Black and Spaulding (*1*) and Hollberg and Armbrust (*2*). The subsequent studies reported by Sanchis and Merrell (*3*) and Baumann and associates (*4–6*) added considerably to the available knowledge of the diatomite filtration process. These latter studies, however, were made on pilot plant or laboratory scale equipment, and relatively little information on actual operating-scale equipment is available. Frazer (*7*) has presented some figures on diatomite usage in the operation of potable-water plants in New York State. Except for these, however, few operating data are available. A number of papers have been published on various phases of filter design and installation (*8–10*), and a number of patents have been issued, primarily for means of backwashing diatomite filters (*11–14*). The author (*15, 16*) has presented a general discussion of the diatomite filtration process.

The work reported here was planned specifically to provide information on the capabilities and limitations of the diatomaceous-earth filtration process. It should be pointed out, however, that the data obtained apply to a single water supply over a relatively short base period. Acknowledging these limitations, data based on actual performance are presented.

Comparison Plants

A detailed description of the combined sand and diatomite filtration processes at the Johns-Manville Research Center was included in a recent article by the author (*17*). Only a brief description, therefore, is included in the present study. Figure 1 is a schematic flow diagram of the combined plants. The sand and diatomite processes can be operated independently or interdependently, as dictated by prevailing conditions.

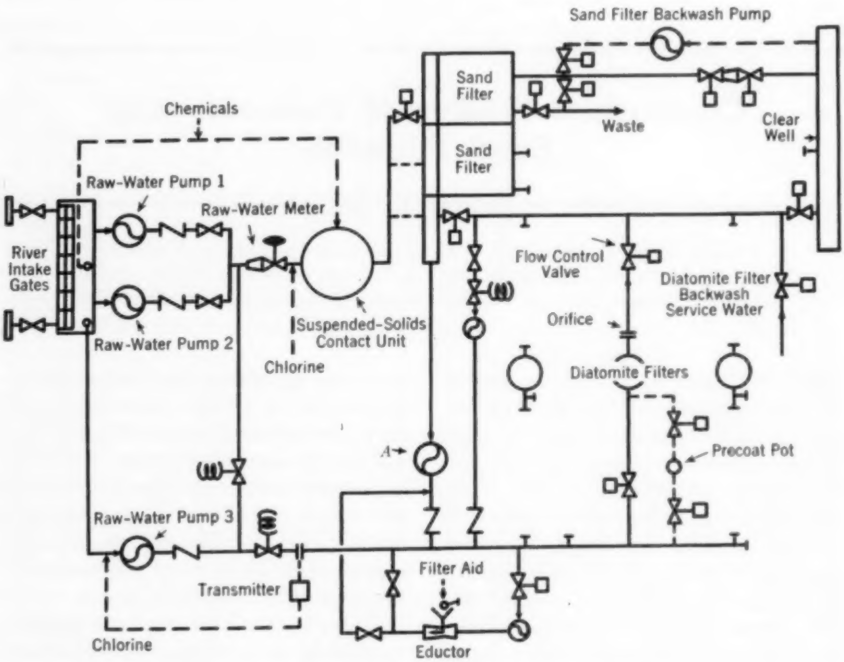

Fig. 1. Flow Diagram of Comparison Study Plant

Centrifugal pumps 1 and 2 lift raw water to suspended-solids contact unit. Pump 3 delivers raw water directly to diatomite filters, or alternatively to suspended-solids contact unit. Chemicals are fed proportionately to flow of raw water by eductor unit which adds chemicals as a slurry each time a set number of gallons passes through the meter. Pump A has dual function of: [1] providing recirculation through precoat pots and filters to apply an initial coating to the diatomite filters before productive filtration begins; and [2] repumping the gravity overflow from the contact unit when diatomite-filtered pretreated water is to be produced in place of diatomite-filtered raw water.

Sand process. The sand filter process, originally installed in 1947, consists of traveling screen, vertical lift pumps, raw-water metering system, chlorinator, slurry pool coagulator with alum and limestone feeders, and two gravity rapid sand filters, each having 250 sq ft of filter surface, a nominal rating of 500 gpm, and a normal working rating of 700 gpm.

Diatomite process. The diatomite process consists of traveling screen, vertical turbine pump, proportioning chlorinator, continuous diatomite feed system, three 150-sq ft filters in parallel, and downstream metering of the filtered water. The nominal rating of the filters is 500 gpm with a normal working rating of 600–700 gpm. The diatomite filters have a closed circuit

bypass system from which an initial coating, or "precoat," is applied, as shown in Fig. 2. A new precoat is applied at the start of every diatomite filtration cycle.

Combined plants. Inasmuch as pretreatment equipment is available and data on the diatomite filtration of pretreated water desirable, an arrangement has been provided which permits the operation of the three diatomite filters as a unit instead of either of the two sand filters. Although the sand and diatomite processes are designed around separate clear well level controls, the combined processes can be operated solely by the sand filter plant controls.

Water Supply

The Raritan River basin is the drainage area for the greater part of central New Jersey. All water received at the filtration plant is pumped from the main stream of the Raritan about ½ mile above its confluence with the Millstone River at Bound Brook. Data are not yet available for the specific period dealt with in this study, but Fig. 3, which covers the period of the original pilot plant work, illustrates the flashy nature of the river. It has a rather broad range of flows and the effect of upstream rains seldom lasts more than 3–4 days and frequently no more than 24 hr.

Between Feb. 1, 1955 and Jan. 31, 1956, turbidity of the raw river water averaged 11 units within a range of 2–800 units (by silica scale determination), and color averaged 28 ppm within a range of 3–400 ppm. During this same period, the minimum flow was 32 mgd and the maximum flow was 15,500 mgd. The latter figure represents one of the highest recorded flows of the river and the former one of the lowest. Considerable industrial and residential building within the area of the Raritan River basin appears to be accentuating the variations in river flow. This results in substantially higher flood levels for the same water volumes encountered only a few years ago.

Another variable which should be noted in connection with the Raritan River is its extreme temperature range. Winter temperatures consistently range close to 32°F and the daily variation is frequently no more than 1°F. Summer temperatures are as high as 95°F with the daily drop as much as 12°F below the daily maximum. The temperature of the river during the warm summer months, when it is at very low stage, is influenced by the return of substantial amounts of industrial condenser–cooling water. As might be expected, such wide variations in temperature result in considerable biological growth, particularly algal forms, and are usually accompanied by a pH shift which follows a daily pattern.

Because of such wide variations in the character of the river a full year was considered to be the minimum period suitable as a data base.

Objectives

In planning the operational program which would provide the comparative data for sand and diatomite processes, it was recognized that two programs are actually possible.

The first program results in a standard operating routine used by the regular plant operators in regularly scheduled operation. The conditions for such routine operation must be based upon earlier experimental operation to determine the most desirable rates of

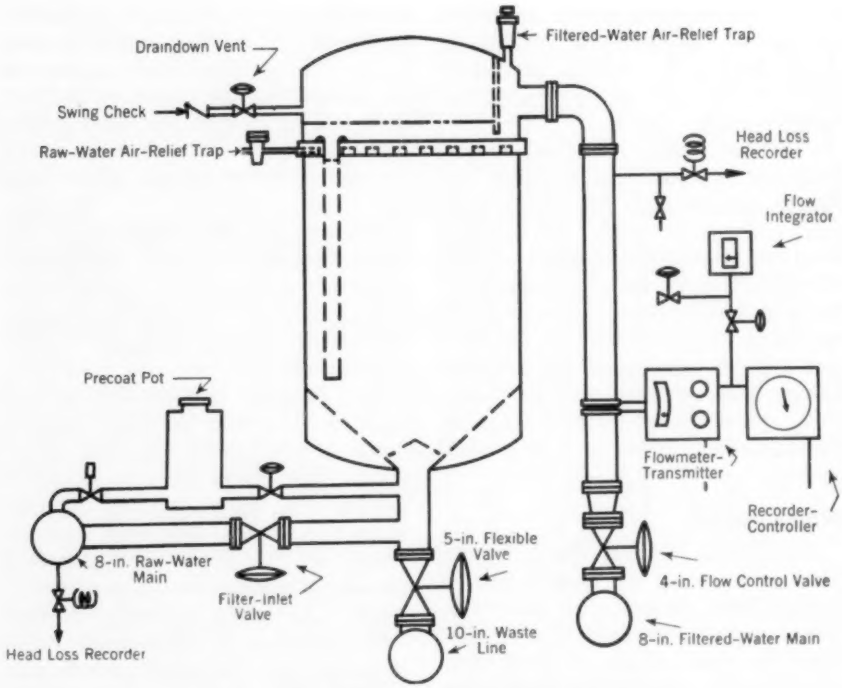

Fig. 2. Individual Diatomite Filter Installation

Schematic elevation drawing shows 8-in. raw-water main at lower left, bypass through precoat pot, normal flow through 4- and 5-in. feed line, passage through filter elements to filtered-water side of system and through flowmeter and controls to 8-in. filtered-water main at lower right. Each of three 150-sq ft filters has identical piping and controls.

flow, rates of filter aid addition, and methods of filter aid addition, as well as to teach the operators the necessary routine. As the sand filter plant had already been in operation for a number of years, no changes were made in operation or operating routine; but in the diatomite plant, where a new process without an established routine was being undertaken, rates which were known to work, based on previous experimental operation, were adopted for routine operation. After adoption, they were allowed to stand unchanged for the duration of the routine operating program.

In addition, a search of the literature reveals a need for information on the effect of rate and similar variables upon operation of the diatomite process. It was proposed that a nonroutine test program also be set up to provide this additional information, with the recognition that there might be some hazard

to the routine evaluation program. This nonroutine program could be accomplished only because the water plant load is extremely heavy during the normal 5-day work week and almost nonexistent on weekends. With a sand filter plant available to handle the load on weekends, the diatomite filters could be scheduled for such purposes as constant-rate filtrations and filter aid evaluations. As plant personnel were required in any case, the cost of such additional experiments would be limited to filter aid, power, chlorine, and maintenance costs.

Such a program was actually set up and operated to obtain nonroutine experimental filtration data. As far as can be determined, the only factor in the routine program which was affected by nonroutine experimental operations was that of maintenance, and specifically the maintenance labor required to open and clean filters when they became dirty. Use of this double program has not only made possible an understanding of the actual volume, costs, and quality of the water produced, but also is permitting determination of what further may be accomplished by the process.

Results of Routine Operations

Establishment of the necessary accounting procedures, installation of necessary meters, and completion of training personnel permitted the beginning of routine operations for comparison purposes on Feb. 1, 1955. The test period was continued through Jan. 31, 1956. During this period normal sand filtration operations were maintained, with the following exception: the clear well level controls were set so that the diatomite filters, when in operation, received demand preference to provide the maximum amount of operating time on stream. Under these conditions the sand filters produced 134.6 mil gal of finished water, with no deduction for backwashing. During a part of the time that the sand filters were in operation, particularly when the Raritan River was muddy, the diatomite filters were operated instead of one sand filter. The diatomite filters under these conditions produced in 58 cycles a total of 24.6 mil gal of finished water before backwashing. The diatomite filters also produced by direct filtration of chlorinated raw water a total of 80.7 mil gal in 173 cycles.

Finished-Water Quality

No visible or measurable color or turbidity was found in the finished water from any of the three processes under routine sampling by water plant personnel using regular water plant methods. All water is prechlorinated to the level necessary to provide a free chlorine residual of 0.2 ppm in the clear well. Under these conditions all tests from all three systems for coliform organisms were negative. Chlorine residual is maintained at 0.2 ppm with the intent of having a zero residual at the points of ultimate distribution, thus protecting such equipment as the powerhouse demineralizer from possible chlorine damage, yet preventing contamination of the system. As the water is not used for drinking, but supplies only development processes, boiler makeup, irrigation, and fire protection demands, such a low residual is adequate. Positive coliform inoculations have occurred with occasional failure of a chlorinator, and decontamination has then been effected by hand feeding of calcium hypochlorite until resumption of gas chlorination.

Finished-Water Cost

Until this study was undertaken there were no means by which the cost of producing water could be determined and set apart from the overall cost of supplying water. The overall figure includes such factors as the cost of high-level pumping and distribution, the cost of labor—whether productive or not (that is, labor required for Saturday, Sunday, and holiday operations when relatively little water is produced)—and various similar overhead categories. Records show that in 1955 the overall cost of water produced through the sand filter system was about $168 per million gallons distributed, and through the diatomite system about $166. Experience has shown, however, that only about 25 per cent of this distributed cost is required to produce finished water in the clear well, as will be shown in some detail later.

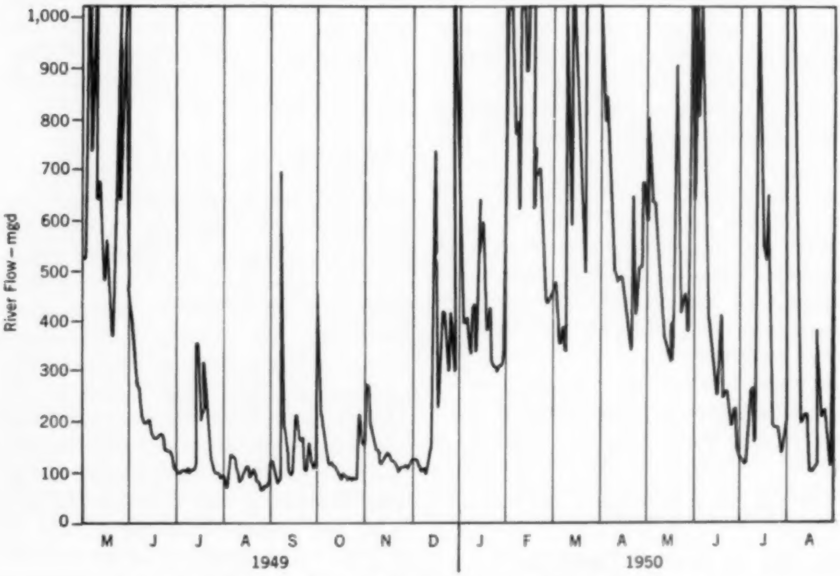

Fig. 3. Stream Flow of Raritan River at Bound Brook, N.J.

Highly variable flows and short duration of effects of upstream precipitation are clearly indicated. Actual stream flows in excess of 1,000 mgd are not shown, but peaks of 15,000 mgd have been recorded.

Comparative Costs

It has been necessary to establish some arbitrary basis for comparison of the sand and diatomite filtration processes. Some costs, such as chemicals, filter aid, and power, can be accounted for directly, but for other costs it has been necessary to assume that each process is concerned only with production of finished water in a clear well. Subsequent distribution of finished water would be the same for all purification processes. Similarly, plant capacity which would require the ultimate utilization of labor is unknown,

but within the range of present operations has been found to be similar in the case of both sand and diatomite filter operations. Thus, this factor too has been stripped from water production costs. The same considerations apply to such items as heating, lighting, and plant supervision. When followed to its end result, this leaves the sand filter process with ultimate costs resulting from initial cost of the plant, cost of chemicals (including chlorine), cost of power, cost of maintenance, and depreciation. Similarly, the diatomite process has ultimate costs resulting from cost of the plant, cost of filter aid, cost of power, cost of chlorine, cost of maintenance, and depreciation. If the diatomite filters are to handle pretreated water, the additional cost of pretreatment equipment and chemicals must also be included. These factors then represent the basic costs for each of the three processes and a comparison of them should provide a defensible economic analysis of all three processes.

From the costs accumulated in routine operations, tables of stripped costs have been prepared for each of the three processes—diatomite filtration of raw water, diatomite filtration of pretreated water, and sand filtration—for which operating data are available. Tables 1–3 show costs for each of the three processes based upon operating data, where possible, and upon detailed engineering estimates prepared to determine the capital investment requirements of each of the three processes.

Power

In comparing these costs it should be noted that, although the chemical and filter aid costs shown are those which have been directly accumulated, the power requirement includes only that part of the metered power which, by calculation, was determined necessary to overcome the resistance of the respective filter media. The method of making this calculation is illustrated graphically in Fig. 4. This power adjustment is based upon the same logic

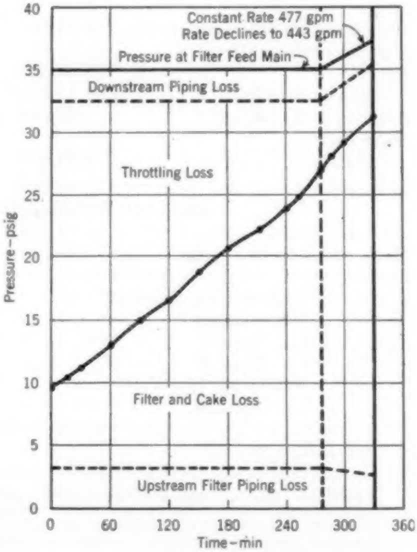

Fig. 4. Power Used by Diatomite Filter

Graphic solution of power consumption within diatomite filter circuit is shown. Data are from a cycle in which a single filter was operated at 477 gpm, resulting in a short cycle and relatively rapid increase in head loss. Solution is similar for diatomite filters in parallel or for sand filters. Ordinate represents pressure available at the filter inlet, which is only a part of the total discharge head (TDH) as determined from the pump characteristic curve. TDH has been corrected for inefficiency of pump and driver. At constant rate, obtained by downstream throttling of the filter, the power use is constant and at any time is the sum of the various components shown. Friction losses were carefully computed, and actual power use is within 10 per cent computed power use.

TABLE 1

*Costs for Diatomite Filtration of Prechlorinated Raw Water**

Estimated Building and Equipment Cost, Maintenance, and Amortization

Item	Cost	Amortization		Maintenance		Annual Expense
		Rate %	Amount	Rate %	Amount	
Raw-water handling equipment	$ 42,261	4	$1,690	†	$1,002	$ 2,692
Pretreatment equipment	—	—	—	—	—	—
Filters	49,523	4	1,981	†	1,159	3,140
Accessories for filters	19,606	4	784	†	474	1,258
Building and storage‡	50,240	2.5	1,256	2	1,005	2,261
Bond issue§	—	—	—	1.5	2,424	2,424
Subtotal	$161,630	—	$5,711	—	$6,064	$11,775

Estimated Chemical, Filter Aid, and Power Cost

Item	Cost/mil gal	Annual Expense
Chlorination chemicals	$ 1.137	$ 637
Filter aid ‖:		
Type A	4.20	2,352
Type B	17.70	9,912
Power ($0.75/kwhr)	1.275	714
Subtotal	$24.312	$13,615
Total Estimated Cost		$25,390

* Based on 2-mgd installed capacity and annual production of 560 mil gal. Costs do not include labor, supervision, building lighting and heating, or cost of installing or operating distribution equipment beyond clear well.

† Where actual costs are not available, maintenance charges have been estimated at a minimum of 1.5 per cent.

‡ Cost of a 40 × 24-ft steel and corrugated asbestos-cement structure, including external clear well, but not including suspended-solids contact basin.

§ Three per cent bond issue on total cost of building and equipment (estimated to average 1.5 per cent of $161,630).

‖ Filter aids used were: Type A, Hyflo Super-Cel; Type B, Celite 503. Both are diatomaceous-earth products of Johns-Manville Products Corp., New York.

as was used in accounting for labor and distribution—that is to say, power used solely for lifting water from one elevation to another or overcoming line friction losses would be required by any process, whereas the concern of this discussion is with those costs which are uniquely assessable to the individual processes.

Buildings and Equipment

The unique-cost approach has also been used in preparing the engineering estimates of plant cost for the respective processes. All equipment required for the functional operation of each process has been included. As the fundamental nature of sand and diatomite filters is different, however, the structures required to house them may also be fundamentally different, and this has been taken into account. For the sand filters a larger structure is required, and conventional reinforced-concrete and brick construction has been used. As diatomite filters are much lighter and more compact, reinforced-concrete foundations and floors, with steel and corrugated asbestos-cement siding and external storage have been used. The life of either type of construction is in excess

TABLE 2

*Costs for Diatomite Filtration of Pretreated Water**

Estimated Building and Equipment Cost, Maintenance, and Amortization

Item	Cost	Amortization		Maintenance		Annual Expense
		Rate %	Amount	Rate %	Amount	
Raw-water handling equipment	$ 42,156	4	$1,686	†	$1,002	$ 2,688
Pretreatment equipment	29,265	10	2,927	†	625	3,552
Filters	49,523	4	1,981	†	1,159	3,140
Accessories for filters	19,606	4	784	†	474	1,258
Building and storage‡	61,240	2.5	1,531	2	1,225	2,756
Bond issue§	—	—	—	1.5	3,027	3,027
Subtotal	$201,790	—	$8,909	—	$7,512	$16,421

Estimated Chemical, Filter Aid, and Power Cost

Item	Cost/mil gal	Annual Expense
Chlorination chemicals	$ 1.78	$ 997
Flocculation chemicals:		
Alum	3.00	1,680
Limestone	1.28	717
Filter aid‖:		
Type A	4.50	2,520
Type C	13.60	7,616
Power ($0.75/kwhr)	1.30	728
Subtotal	$25.46	$14,258
Total Estimated Cost		$30,679

* Based on 2-mgd installed capacity and annual production of 560 mil gal. Costs do not include labor, supervision, building lighting and heating, or cost of installing or operating distribution equipment beyond clear well.

† Where actual costs are not available, maintenance charges have been estimated at a minimum of 1.5 per cent.

‡ Cost of a 40 × 24-ft steel and corrugated asbestos-cement structure, including suspended-solids contact basin and external clear well.

§ Three per cent bond issue on total cost of building and equipment (estimated to average 1.5 per cent of $201,790).

‖ Filter aids used were: Type A, Hyflo Super-Cel; Type C, Celite 535. Both are diatomaceous-earth products of Johns-Manville Products Corp., New York.

of 40 years, by which time obsolescence will probably be a more significant factor than the structural life of the buildings. As the plant upon which the data were accumulated includes a slurry pool coagulator, similar equipment external to the main buildings was included in the estimates where applicable. Prices used for all the estimates were those in effect in October 1955, based on actual quotations at that time.

Amortization and Depreciation

For purposes of computing the cost of the invested capital a return of 3 per cent, the common current rate for municipal bonds, has been used. For the purposes of these calculations continuing amortization of the declining-balance type results in an actual overall interest rate of 1.5 per cent, which is the figure actually appearing in the tables. Depreciation, on the other hand, is based upon the best estimate

TABLE 3

*Costs for Sand Filtration of Pretreated Water**

Estimated Building and Equipment Cost, Maintenance, and Amortization

Item	Cost	Amortization		Maintenance		Annual Expense
		Rate %	Amount	Rate %	Amount	
Raw-water handling equipment	$ 57,140	4	$ 2,286	†	$ 857	$ 3,143
Pretreatment equipment	40,995	10	4,100	†	625	4,725
Filters	11,445	2	229	†	172	401
Accessories for filters	11,726	4	469	†	176	645
Building and storage‡	175,285	2.5	4,382	2	3,506	7,888
Bond issue§	—	—	—	1.5	4,449	4,449
Subtotal	$296,591	—	$11,466	—	$9,785	$21,251

Estimated Chemical and Power Cost

Item	Cost/mil gal	Annual Expense
Chlorination chemicals	$1.780	$ 997
Flocculation chemicals:		
Alum	3.00	1,680
Limestone	1.28	717
Power ($0.75/kwhr)	0.675	378
Subtotal	$6.735	$ 3,772
Total Estimated Cost		$25,023

* Based on 2-mgd installed capacity and annual production of 560 mil gal. Costs do not include labor, supervision, building lighting and heating, or cost of installing or operating distribution equipment beyond clear well.
† Where actual costs are not available, maintenance charges have been estimated at a minimum of 1.5 per cent.
‡ Cost of a 40 × 40-ft brick and reinforced-concrete structure, including suspended-solids contact basin and clear well beneath building.
§ Three per cent bond issue on total cost of building and equipment (estimated to average 1.5 per cent of $296,591).

of the life of the individual capital items. For many of these items, such as diatomite filters, there are not yet accurate depreciation figures, but for most other items standard tables have been used (*18*). Where applicable, US Bureau of Internal Revenue codes have also been used. Every effort has been made to assure that these figures are conservative.

Maintenance

Determination of maintenance costs presents some unexpected problems. In a new plant, especially one in which the process and equipment are new or unique in terms of general industry practice, maintenance charges derived from minor alterations necessary to make the process work smoothly, or from cleaning up the mess resulting from operator training errors, may cause an appreciable increase in annual maintenance charges. After such an initial period, however, maintenance charges would be expected to level off and remain relatively constant except for expected occasional maintenance on heavy pieces of equipment. Such has been the experience with the sand filter plant being used for comparison. The plant has been in service for 9 years during which no more than routine maintenance has been required until

this year. Now the flocculating equipment, sand filters, valves, actuators, and operating tables are undergoing thorough renovation. Sand filter maintenance cost therefore has been computed as annual accumulated maintenance charges plus a prorated portion of the major overhaul expense.

With diatomite filters the experience has been somewhat different. There was a period of higher initial expense such as is typical of any new plant, and most of this expense was incurred before the base period used for computation in this report. But these diatomite filters seem to require a continuing higher maintenance, largely because of the need for periodic opening and cleaning of the filters themselves. More than half the accumulated maintenance charges shown for diatomite filters are caused by this single factor. The remainder of the diatomite filter system consists of standard components, such as pumps, valves, and pipe, for which normal maintenance requirements have been experienced. Maintenance costs on the diatomite filters themselves may be reduced through the use of other septum materials, other backwashing techniques, or better filter-cleaning techniques. The costs shown are those actually accumulated during the test period. As it is presently impossible to differentiate between maintenance costs attributable to diatomite filtration of raw and pretreated water, identical costs have been assumed for each process.

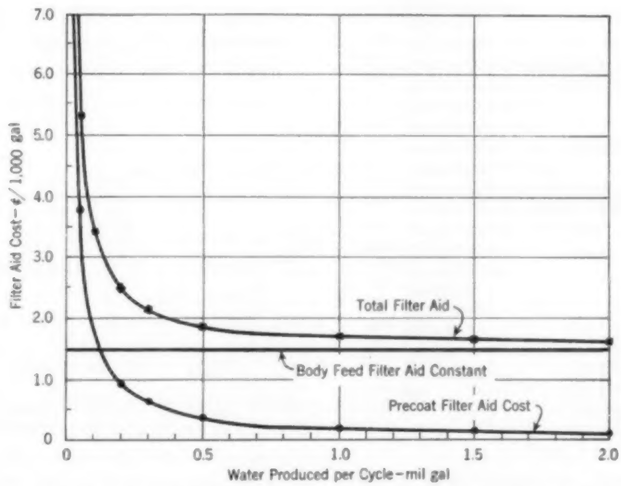

Fig. 5. Comparison of Precoat and Body Feed Filter Aid Cost

Precoat cost is amortized over the total volume of water to be filtered, while the continuous feed level remains relatively unchanged. Cycles shorter than 250,000 gal (based on 50 lb per precoat application) cause a high filter aid cost.

Chlorination

Chlorine for prechlorination has been shown as a separate item because, although both sand and diatomite sys-

tems have the same residual in the finished water, only about 60 per cent as much chlorine is required when diatomite filters are filtering raw water directly. Chlorine demand of the raw water varies seasonally as well as with the daily condition of the river and, as might be expected, the difference in chlorine requirements of the two systems fluctuates in a similar manner. The diatomite process, however, may require as little as 45 per cent of the chlorine required by the sand filter process and seldom exceeds 75 per cent. The additional chlorine requirement of the sand filter plant is about equally expended in the flocculating equipment and in the sand beds themselves. It might be expected that some chlorine saving would result from the diatomite filtration of pretreated water, but experience to date has been too limited to establish this point definitely.

Chemical costs other than chlorine represent actual costs accumulated during the pretreatment of 159.2 mil gal during the test period. These costs are assumed to be identical whether the pretreated water is to be filtered by sand or by diatomite. These chemical costs are comparable to those obtained in the operation of the sand filter plant over the past several years.

Filter Aid

The major operating cost of diatomite filters is the filter aid itself. The costs shown are those actually accumulated during the filtration of 80.7 mil gal of raw water and 24.6 mil gal of pretreated water. No attempt was made to regulate filter aid usage closely. On the contrary, the opposite philosophy—that enough filter aid should be used to keep the station operating smoothly—was adhered to. There is little question that better filter aid costs could be achieved, but costs shown are those actually obtained.

TABLE 4

Filter Aid Use for Raw-Water Filtration

Cycles	Throughput *mil gal*	Precoat Filter Aid *tons*	Precoat Cost *$/mil gal*	Body Feed Filter Aid *tons*	Body Feed Cost *$/mil gal*	Total Filter Aid Cost *$/mil gal*
All cycles	80.722	4.416	4.20	16.877	17.70	21.90
Cycles of less than 0.25-mil gal throughput	4.759	0.842	13.56	1.158	20.61	34.17
Cycles of more than 0.75-mil gal throughput	26.373	0.550	1.60	3.995	12.83	14.43
All cycles less short cycles	75.963	3.575	3.61	15.720	17.53	21.14

In discussing filter aid costs it should be recognized that filter aid is used in two ways: [1] as *precoat,* in which a definite weight of material is applied to the filtering surface prior to the actual start of production and [2] as *body feed,* in which a continuous feed of filter aid is maintained proportional to the amount and nature of the suspended solids to be removed. Figure 5 illustrates the effect of these separate filter aid usages. It is particularly im-

portant to note that the cost of the precoat must be amortized over the total volume filtered, which means there is a minimum economic cycle length, probably about 250,000 gal. Body feed, on the other hand, remains relatively constant for each thousand or million gallons filtered. Total filter aid cost is, of course, the sum of the precoat and body feed costs for any given cycle and it seems apparent that once the minimum economic limit has been reached only relatively minor filter aid economies can result. Other economies (such as in power consumption and capital investment costs) resulting from long cycles will be discussed later.

As a first step in analyzing the filter aid performance of the diatomite filters the bar graphs shown as Fig. 6 and 7 were prepared, the first to illustrate the filtration of raw water, the second, of pretreated water. A broken line sets off cycles in which less than 250,000 gal was produced, Fig. 5 having suggested this production volume as the minimum at which precoat filter aid costs became reasonable. A second broken line sets off cycles which produced more than 750,000 gal.

These cycles which are uneconomically short represent various kinds of failures such as valve failures, filter aid feeder failures, filter aid failures, operator errors, and failure to recognize the need to have the filters opened and cleaned. Conversely, the extremely long cycles represent an ultimate objective which will be reached on a regular basis only when presently unsolved problems, such as the attainment of 100 per cent backwashing effectiveness, have been achieved. The majority of the runs which lie between the two extremes may be described as present normal operation.

To illustrate the effect of cycle length upon operating costs, Table 4 has been prepared. Using the data for all cycles as a norm, the predicted cost of precoat filter aid for short cycles does substantially increase the total filter aid cost. Similarly, filter aid cost for very long cycles decreases very little because of the additional cycle

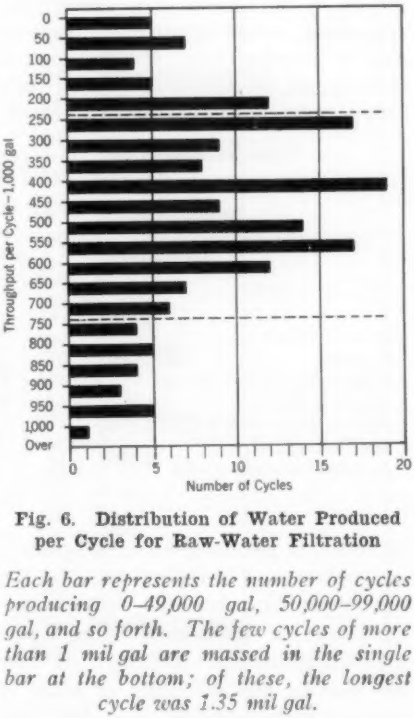

Fig. 6. Distribution of Water Produced per Cycle for Raw-Water Filtration

Each bar represents the number of cycles producing 0–49,000 gal, 50,000–99,000 gal, and so forth. The few cycles of more than 1 mil gal are massed in the single bar at the bottom; of these, the longest cycle was 1.35 mil gal.

length. In the last line of the table costs shown are those from which the cost of the short cycles has been excluded. These might be regarded as a better basis for future performance than the figures actually used, in which all filter aid consumption was included.

For diatomite filtration of pretreated water, it will be noted in Table 5 that

filter aid costs are very nearly as high as for the filtration of raw water. This would not normally be expected. As with raw-water filtration (*see* Table 4), filter aid costs for pretreated-water filtration were found to vary with cycle length. In addition, a factor not previously discussed is applicable to pretreated-water filtration—the amount of floc carryover which must be removed by the filters. The slurry pool coagulator, when operated in balance and at constant rate, is capable of a high degree of pretreatment with very little carryover. Such equipment, however, is generally affected by thermal upsets, and in the present case the equipment also has the further disadvantage of necessarily being operated intermittently. During cold winter months the solubility of alum is poor, with consequent "pinpoint" floc formation or after-precipitation. In the warm summer months the wide range of river water temperatures results in an almost predictable daily boilup with consequent heavy carryover. On rare occasions when the equipment can be run continuously for several hours, operation has begun to level out and, if continued, presumably would become quite stable in time. Normal operating cycles, however, are about 1 hr and there are only occasional operating periods as long as 2 hr. For the accumulation of the data now being discussed it was decided to carry a sufficiently high body feed rate to insure pretreated water filtration except in the most severe cases of floc carryover. Referring again to Fig. 7, the cycles in which throughput was greater than 750,000 gal represent periods of unusually stable pretreatment operation, with some continuous filter aid feed rates of less than half the average value. Now that the flocculating equipment is newly reconditioned, it is hoped that additional experience with pretreated water in the diatomite filters will provide a sounder economic basis than that presented here.

TABLE 5

Filter Aid Use for Pretreated-Water Filtration

Cycles	Throughput *mil gal*	Precoat Filter Aid *tons*	Precoat Cost *\$/mil gal*	Body Feed Filter Aid *tons*	Body Feed Cost *\$/mil gal*	Total Filter Aid Cost *\$/mil gal*
All cycles	24.565	1.441	4.50	3.803	13.60	18.10
Cycles of less than 0.25-mil gal throughput	2.368	0.449	14.56	.449	16.65	31.21
Cycles of more than 0.75-mil gal throughput	8.658	0.200	1.77	1.268	12.84	14.61
All cycles less short cycles	22.197	0.992	3.43	3.354	13.25	16.68

When all the costs which are reflected by the foregoing variables are tabulated (*see* Table 6) for diatomite filtration of raw water, diatomite filtration plus pretreatment, and sand filtration plus pretreatment, the resulting costs per million gallons are \$45.71, \$55.22, and \$45.31, respectively. For diatomite filtration of raw water and sand filtration plus pretreatment the

figures are, for all practical purposes, equal. The cost of diatomite filtration plus pretreatment reflects the high body feed cost encountered. The intent has been to make the diatomite figures maximal both as to operating costs and investment costs, and it seems probable that the best opportunities for cost improvement lie with the two diatomite processes discussed.

Other Investigations

As suggested earlier, before the plant was put into operation a number of phases in the diatomite filtration process received attention. Within the 1-year base period others were investigated on a purely experimental basis. Included were such items as improved methods for adequate intermixing of the filter aid, methods of filter aid feeding, degree of clarification effected by filter aids of various permeabilities, effect of filter rate on volume of water produced per cycle, causes of filter element fouling, and improved methods of cleaning fouled elements. Some of these are continuing studies and as reported herein should be considered progress reports.

Filter Aid Mixing

One of the first problems encountered in the operation of the diatomite filters was the need for adequate mixing of the continuous filter aid feed throughout the entire volume of water to be filtered. In the filter station, all water to be filtered comes to the individual filters through a single 8-in. main. Originally filter aid was introduced in slurry form at a point some 5 ft above the first filter takeoff. Although a number of differently designed injection nozzles and distributors were tried, none succeeded in producing a sufficiently uniform distribution within the main to cause identical performance of all three filters. Subsequently, a system was set up in which the filter aid is injected as a slurry into the center of the main some 20 ft

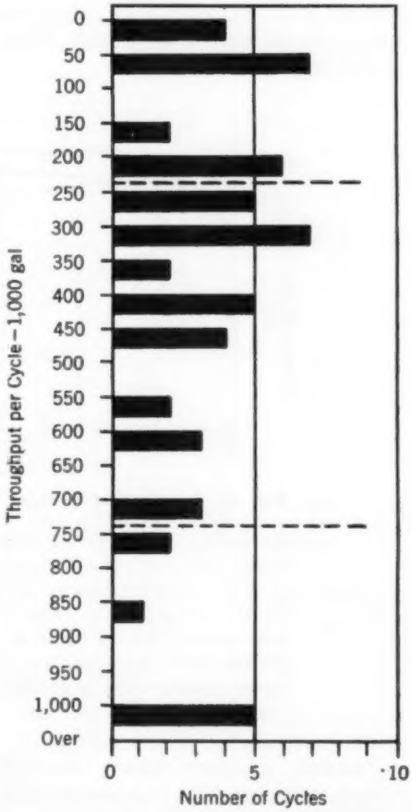

Fig. 7. Distribution of Water Produced per Cycle for Diatomite Filtration of Pretreated Water

before the filters, the intervening pipe including one 90-deg ell, one 45-deg ell, and a tee, all of which provide turbulence. Injection is accomplished through a 1-in. bend which is faced with open end toward the source of

flow. At 600–700 gpm through the 8-in. main, flow through this injector averages 25–30 gpm. Since this system was installed, no difficulties have been encountered that could be traced to lack of adequate mixing of the filter aid. This is true even though a number of different grades of filter aid have been injected over a wide range of flow rates.

installation can be very serious because of the intermittent nature of the operation. It was therefore decided at the outset that some system of dry feeder combined with an eductor should be attempted. Initially, a dry feeder of the rotating-vane type, with both variable rate of rotation and an adjustable opening to control the rate of feed, was installed. This feeder was finally

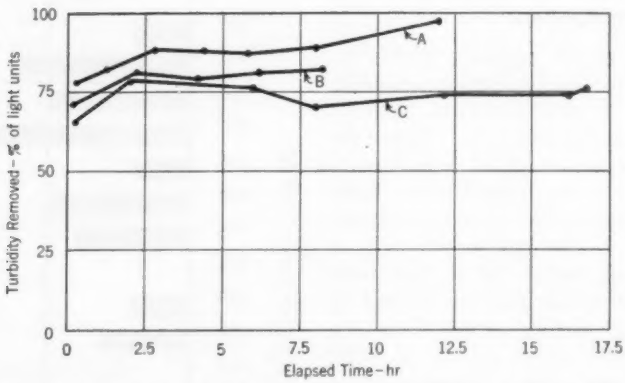

Fig. 8. Removal of Turbidity by Three Grades of Filter Aid

Turbidity, which is an optical approximation of suspended solids and colloidal material, was determined by Tyndall meter. Unfiltered-water turbidity was low, averaging 5 units on the silica scale. Higher raw-water turbidities result in substantially higher removal efficiencies but comparative data for the three filter aids are not available. Sag in the Type C curve results from a decrease in raw-water turbidity. Each type of filter aid was used for both precoat and body feed.

Another problem which received considerable attention because of early difficulties it caused was that of metering a dependable continuous filter aid feed. At other company locations where diatomite filters are installed, agitated slurry tanks and various types of slurry pumps are used for the injection of the filter aid into the system. Such pumps require considerable maintenance and, more important, are subject to stoppages which in the present abandoned when it was found to have two serious deficiencies: [1] it caused different grades of filter aids to be fed at widely different rates varying by as much as 10:1, and [2] it seriously damaged the structure of some filter aids, particularly the more permeable ones (which are also the more expensive grades). The next dry feeder tried was of the vibratory type, but it too provided widely variable feed rates for different grades of material and

sometimes wide variations within a single grade of material. For more than a year a new type of feeder which combines the functions of a vibrator and a screw has been in use for all diatomite filter operations (*19*). This feeder has effectively controlled the rate of filter aid feed and has permitted a high degree of reliance on the filter aid feed values.

The filter aid feeder drops the metered filter aid (the normal feed rate for raw water is about ¼ lb per minute) into a flooded funnel which is kept full of water by a float valve. The filter aid is dropped on an 8-mesh brass screen through which it is washed by the water entering the funnel. The screen protects against dropped objects or foreign material which might block the relatively small eductor openings. The filter aid, now in slurry form, is drawn into the suction of a 1-in. eductor which is powered by pressure from the filter feed main and a small booster pump. Slurry picked up by the eductor is fed into the main filter feed line as described earlier. The only feature of the eductor which is unique is the design of the jet nozzle, for which a stock 1-in. cast-iron body is used. The nozzle itself must be designed for the conditions prevailing in a given installation, but maintenance is extremely low and to date no nozzle has shown appreciable wear; even after 6 months the cast-iron bodies are serviceable.

TABLE 6

*Comparison of Diatomite and Sand Filter Stripped Costs**

Item	Diatomite Raw Water	Diatomite Pretreated Water	Rapid Sand Pretreated Water
Installed Cost of Plant	$161,630	$201,790	$296,591
Operating Costs			
Chlorine	637	997	997
Flocculating Chemicals	—	2,397	2,397
Filter Aid	12,264	10,136	—
Power	714	728	378
Maintenance	3,640	4,485	5,336
Investment Costs			
Amortization	5,711	8,909	11,466
Interest on Bonds	2,424	3,027	4,449
Total Cost	$25,390	$30,679	$25,023
Cost/mil gal	45.34	54.78	44.68
Cost/mil gal corrected for backwash losses†	45.71	55.22	45.31

* Based on 2-mgd installed capacity and annual production of 560 mil gal.
† Backwash losses for diatomite, 0.8 per cent; for sand, 1.4 per cent.

The present combination of dry filter aid feeder, booster pump, and eductor has required remarkably little maintenance and appears to be sufficiently reliable for general use.

Permeability of Filter Aids

The next phase of diatomite operation to receive attention was selection of the proper grade or grades of filter aid. Diatomite filter aids are commercially available in about ten different permeabilities or flow rates, of which

the four or five most permeable grades appear to be applicable to water clarification. As permeability is a function of the size of filter aid particles and their interstices, it seems apparent that use of more permeable filter aids usually will be accompanied by decreased filtrate clarity. Of course, if all suspended solids to be removed are coarse, a more permeable filter aid can be used without sacrificing clarity. Conversely, some semicolloidal solids are so fine that they can be removed only by the least permeable filter aids. In most surface waters, suspended solids present a broad and variable range of particle sizes; hence, some particular grade of filter aid will provide optimum removal.

To determine which grades of filter aid are best adapted for Raritan River water several series of filter runs were made, with the results shown in Fig. 8 and 9. Both raw- and filtered-water samples were evaluated by a Tyndall meter *(20)*, which uses the scattered light picked up at 90 deg related to the incident light as a measure of suspended solids. If a sample of raw water contains 100 particles evenly ranged between coarse and fine, each of the suspended particles is capable of reflecting some light at 90 deg and the total value of this light may be arbitrarily set at 100 light units. Filtration of this same raw water may remove the 75 largest particles with a resultant reduction in the amount of reflected light measured at 90 deg. After filtration, the light value may be reduced to 20–25 light units, but the *weight* of suspended solids remain-

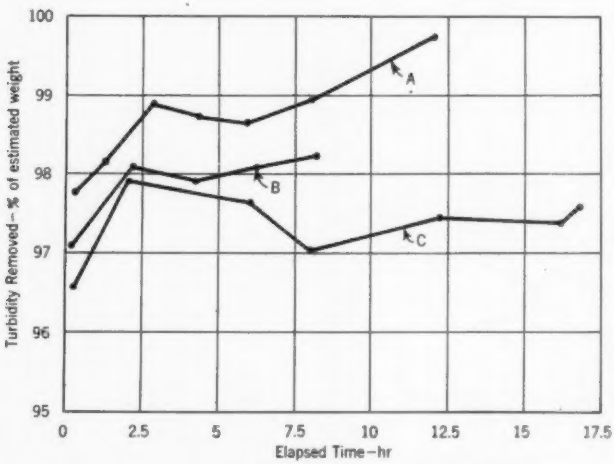

Fig. 9. Estimated Removal of Suspended Solids by Three Grades of Filter Aid

Removal is higher than in Fig. 8 because all coarse material, which represents most of the weight of the suspended solids, has been removed.

ing is far less because the larger and heavier particles were removed by filtration. It is estimated that the weight factor for the remaining light units of the results shown in Fig. 8 is only about 10 per cent. If these results are then replotted (as in Fig. 9) as percentage of weight removed, a better measure of removal efficiency is provided.

It is interesting to note that all three filter aids * shown have a somewhat lower initial-removal efficiency and several hours are required to reach a stable removal level. With Type C, the most permeable of the three grades used, the subsequent fluctuations in removal efficiency resulted largely from fluctuations in raw-water quality rather than filtered-water quality; that is, a *decrease* of suspended solids in the raw water which did not at once produce a corresponding *decrease* of suspended solids in the filtered water would produce an apparent decrease in the percentage of material removed, even though the filtered-water quality might not have decreased. Continuing studies indicated that Type C was particularly susceptible to raw-water variations, and Type B was therefore selected as the regular body feed grade for raw-water filtration. To overcome the lower initial clarification with Type B, less permeable Type A is always used for the initial precoat. This arrangement has the added advantage of providing the filtering elements with the least penetrable coating.

Filtration of pretreated water usually involves the removal of only relatively coarse floc carryover. As might be expected, a relatively permeable grade of filter aid can be used for pretreated water. Floc trapped in the filter cake remains active, just as it does in a sand filter, and removal efficiencies are extremely high, even though pretreatment may be partially ineffective. In diatomite-filtered pretreated water, concentration of residual suspended solids is below the limits of detection. While some sand-filtered samples are equally well clarified, this has not been true consistently, and during cold periods when solubility of alum is a real problem there is frequent, though seldom serious, carryover. For pretreated-water filtration through the diatomite filters, Type C is used for continuous body feed. To reduce to a minimum the possibility of floc penetration of the precoat, Type A is also used for this initial coating.

Filtration Rates

The question of what filtration rate is the most economical for diatomite filtration of water is a perennial one. When the present plant was designed, some flexibility was specifically provided to permit studies of this important point. Provision was made for control of flow rates through the range of 0.5 to more than 3 gpm per square foot. In routine operations, the filters operate at 600–700 gpm or 1.3–1.5 gpm per square foot, as experience has determined that this is a "comfortable" rate. Experimental operations were undertaken at various other rates, however, and under conditions which avoided the complications produced by intermittent operation. Three series of runs were made with three different grades of filter aid during a period in

* Diatomaceous-earth filter aids used were: Type A, Hyflo Super-Cel; Type B, Celite 503; and Type C, Celite 535. All are products of Johns-Manville Products Corp., New York.

which the river flow was relatively stable; hence the results are thought to be comparable. These series are shown in Fig. 10. It can be seen that decreases in rate bring about substantial increases in throughput per cycle. In line with previous discussion, decreases in rate thereby bring about decreases in filter aid costs, particularly of the precoat filter aid. It should be pointed out, however, that decreases in rate also represent increases in capital investment cost.

Actually, the results of the preceding three series of experimental runs were disappointing, as they did not correlate with earlier pilot plant experience. These latest experimental runs appeared to be influenced by partial fouling of the filter elements, which of course meant a decrease in usable filtering surface and corresponding increase in actual rate per square foot. To check this, filters were torn down and thoroughly cleaned by hand and reassembled, after which another series of experimental runs was undertaken with Type B filter aid. The results of this last series of runs are shown in Fig. 11, together with the rescaled curve for the same filter aid taken from Fig. 10. There seems little question that the partial fouling of the elements had a substantial effect on the earlier experiments. If it is assumed that the earlier Type C runs were made with a 30 per cent loss in area, which corresponds to a 43 per cent increase in filtering rate per square foot of available area, it will be found that the earlier curve approximately coincides with and satisfactorily extends the curve made by data from the clean filters.

From these results it is apparent that, although the rate per nominal square foot of filtering area is important in the design of a diatomite filtration station, the problem of keeping the filters free from fouling is of equal or greater significance. It is also apparent that operating cost resulting from partially fouled filters is not limited to the cost of opening the filters and manually cleaning them. Indeed, most of the very long cycles in Fig. 6 and 7 were obtained after the filters were cleaned. The importance of this factor is recognized and a program is

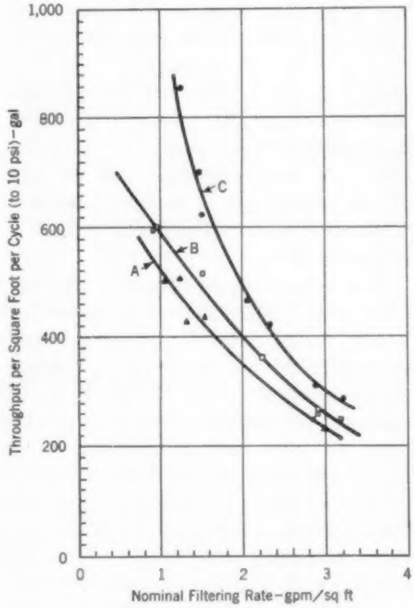

Fig. 10. Relationship of Filtering Rate to Throughput

Three filter aids, used for both precoat and body feed on raw water, have a related total throughput per square foot, at a terminal net head loss of 10 psi, generally in proportion to the individual filter aid permeabilities. These curves might be expected to break back toward the ordinate at very low rates but such rates are impractical for the equipment used.

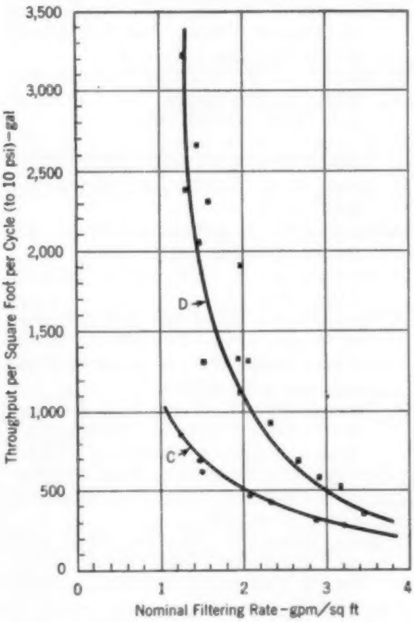

Fig. 11. Effect of Filter Septum Condition on Throughput

Curve C has been rescaled from Fig. 10. Curve D resulted from a series of filtering cycles run after the filters were newly cleaned. Adverse effect of element fouling is readily apparent.

now being established to test not only other types of filter septum materials but also entire filters of different design in an effort which could substantially improve the cost structure of diatomite filter operations.

Another phase of this program is an investigation of chemical cleaning methods for diatomite filters with the same objective of reducing costs.

Conclusions

The present study was undertaken for the specific purpose of providing a basis for the comparison of the diatomite filtration and conventional sand filtration for the clarification of water. A period of 1 year of operation for each type of plant on the same water supply has been used as a basis of comparing operating costs. Other charges related to the nature of the processes have been calculated either from actual maintenance figures or from reproduction costs of plants of identical capacity considering only the water clarification processes. Based on these estimates, the cost of diatomite filtration of raw water is approximately equal to the cost of sand filtration plus pretreatment. The costs of diatomite filtration plus pretreatment under the prevailing conditions were found to be somewhat higher.

Certain key points in the diatomite filtration process have been discussed and evidence has been presented indicating that substantial improvements in the cost picture are possible. This is probably more nearly true of the relatively new diatomite process than of the conventional sand filtration process, which has already reached a high state of development.

Water produced by the diatomite filtration process, whether prechlorinated raw water or prechlorinated pretreated water, has acceptable clarity and has consistently met bacteriological standards with no more than 0.2-ppm chlorine residual.

The combined-filter installation remains in operation and additional data are being accumulated. Furthermore, attention is being given to specific phases of filter design and filter operation from which it is expected that additional factual information will become available. It is hoped that other operators will in time provide correlative data.

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Economics of Sludge Removal From Rectangular Basins at Chicago

Fred G. Gordon

A paper presented on May 7, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by Fred G. Gordon, Asst. Chief Engr., Bureau of Eng., Dept. of Public Works, Chicago, Ill.

THIS article reports a study made at Chicago of the cost of sludge removal from filter plant settling basins and the comparative effects of manual and mechanical methods on plant operation.

The three rectangular settling basins of the Chicago South District Filtration Plant are equipped with sludge scrapers serving one-third their total area. In the areas not served by scrapers, sludge is removed by hand, usually in the spring and fall. The scrapers were installed about 2 years after the plant was placed in operation. Prior to their installation the basins were cleaned three times a year. Because only two-thirds of the settling-basin capacity is available when one of the basins is being cleaned, shorter detention time and higher velocities are required, adversely affecting the operation of the entire plant. More chemicals are used, and filter runs are shortened. Experience with the scrapers now in service has shown a major reduction in the annual inoperative period of each basin. Installation of additional scrapers to serve the unequipped area of the basins would still further reduce the inoperative period of each basin and would eliminate most of the hand-cleaning costs. Opposed to these various advantages are the maintenance costs and debt service charges for the equipment. Although this study covers an admittedly special situation, certain of the data presented may be useful in verifying sedimentation-basin performance and its effect on plant operation.

Physical Plant

The three settling basins are double-decked parallel-flow structures. Each basin is 500 ft long and 138 ft wide. The height of the lower basin from the top of the lower floor to the underside of the intermediate floor is 16.08 ft. The overflow in the upper basin is 16.54 ft above the average elevation of the intermediate floor. Both floors slope 1 ft from side wall to side wall. In the lower basin a collecting channel runs the length of one side wall. The first eight bays of each basin are equipped with sludge scrapers. Each of the eight scrapers serves both decks by traveling transversely across the upper deck to a chute leading to the lower deck and then across the lower deck to the collecting channel at the side wall. Cross collectors in this channel move the sludge to a sump from which it is pumped to the discharge point. During manual cleaning, sludge is hosed to the collecting channel.

Water is conditioned for settling in double-decked mixing channels in which the direction of flow is parallel to the horizontal shaft of the rotating

mixing paddles. Detention periods and nominal velocities at the plant rating of 320 mgd are shown in Table 1.

Plant Operation

The hydraulics of the tunnel system connecting the plant and the three South District pumping stations make it desirable to maintain a high level in the filtered-water reservoir. This is accomplished by the use of automatic controllers on filter units. The filters therefore—with the exception of certain fixed-rate units—follow the demand very closely. The low-lift pumps, operated without throttling, follow the demand by steps or blocks. The difference between the demand by the filters and the pumpage is absorbed in the mixing and settling basins and their connecting channels. On a typical hot summer day there may be three pump changes within 24 hr, with a variation of water level in the filtered-water reservoir of 1 ft and a variation of less than 2 ft in the settling basins. On such a day the low-lift pumpage between 11:30 PM and 6:30 AM may be at a 255-mgd rate, or 70 per cent of the average rate for the day; between 6:30 AM and 10:30 AM, at a 370-mgd rate or 102 per cent of the daily average; and for the remainder of the day at a 420-mgd rate, or 115 per cent of the daily average. Such a schedule results in wide variations in settling-basin velocities in short intervals of time.

TABLE 1

Detention Periods and Velocities at Plant Rating of 320 mgd

Basins in Service	Detention Time *hr*	Velocity *fpm*
3 settling	3.6	2.32
2 settling	2.4	3.48
3 mixing	0.84	14.34
2 mixing	0.56	21.51

Since 1951 the average daily pumpage for the year has exceeded the nominal plant rating of 320 mgd. For 1955 this was 354.8 mgd. The maximum day's pumpage, which occurred in 1953, was 488 mil gal. The rate on the maximum hour of the maximum day was 546 mgd.

Results of Scraper Installation

The delay in the sludge scraper installation permitted a before-and-after comparison to be made of the amount of sludge accumulated between cleanings.

Observations were made of undrained and drained sludge depths whenever a basin was cleaned. The data on the undrained sludge depths were obtained by measuring markings on the columns which indicated depths of sludge when the basin was full of water. Table 2 shows average observations over a 6-year period.

In general it was found that undrained sludge depths in the upper basins were less than in the lower basins by about 15 per cent, presumably due to the varying water level in the upper basins. This was true both before and after the installation of sludge scrapers over the first one-third of the area of both basins. The drained sludge, however, did not follow this relationship. Before installation of the equipment, the drained sludge depths in the upper basins were less than those in the lower basins by 52 per cent. After installation of the equipment, the drained-sludge depths in the upper basins were less than those in the lower basins by 38 per cent. These figures would indicate a sludge of higher water content in the upper basins than in the lower, but with an improved relationship after the equipment installa-

tion. Both drained and undrained sludge showed a quite uniform tapering off in depth from scraper area to outlet. After the installation of scrapers, the depths at the outlet were approximately one-third of those adjacent to the area served by the scrapers.

Additional data on sludge accumulation is furnished by Mrva (*1*), who compares three cleanings in 1946 (prior to scraper installation) with two cleanings in 1952. His data are summarized in Table 3. Mrva found that prior to the scraper installation the volume of drained sludge was 0.5 cu yd per 1 mil gal of water passed through the basins. After the installation, the amount of sludge in the area not served by scrapers dropped to a little less than 0.16 cu yd per 1 mil gal of water. On the installation of scrapers the direct labor required for cleaning basins dropped from 47.6 to 19.0 man-hr per 1,000 mil gal passing through the basins.

Shutdown Time

The total cleaning period for the three basins has averaged 74 days per year. This time would be materially reduced—but not eliminated—if scrapers were installed in the entire settling-basin area. Maintenance on the slow- and rapid-mix equipment and on the sludge scrapers would require shutting down each basin at least once a year. It is probable that 5 or 6 days per basin would be required to dewater, perform the necessary inspection and maintenance work, partially fill the basin with heavily chlorinated water, and then backflush with treated settled water.

TABLE 2

Average Sludge Depths Before and After Scraper Installation

Distance From Inlet *ft*	Depth of Sludge—*ft*			
	Before Installation*		After Installation†	
	Undrained	Drained	Undrained	Drained
Lower Basins				
20	6.2	3.3	‡	‡
160	7.1	3.6	5.2	2.5
340	4.9	2.3	3.9	1.6
460	3.9	1.6	1.8	0.8
Upper Basins				
20	5.1	1.8	‡	‡
160	5.7	1.5	4.8	1.7
340	4.8	1.1	2.8	0.9
460	3.2	0.7	1.5	0.5

* Maximum period covered: August 1945–March 1948. Average number of days between cleanings was 128.
† Maximum period covered: January 1948–November 1951. Average number of days between cleanings was 176.
‡ Location of scraper.

If this total period were assumed to be 14 days, then, with a complete scraper installation, shutdown time for all basins would be reduced 60 days per year.

Complete Mechanical Removal

In forecasting savings with complete sludge removal equipment, estimates are required of the reduction in labor, chemicals, and wash water costs. The labor item may be determined from data already presented.

The chemical and wash water savings can be estimated by comparing operating results when two basins only are in service with normal operation immediately following cleaning. For this analysis two 7-day periods—one at the beginning and the other at the end of the cleaning period—were averaged to determine performance with two basins. For three basins, a 7-day period following resumption of normal operations was used.

Statistical data cover the years 1953–1955, during which period the chemical treatment employed (split treatment using aluminum sulfate in two basins and ferrous sulfate in the remaining basin) was fairly uniform. The 42 days per year considered (or a total of 126 days for the 3 years) tend to minimize the effects of such variables as microorganisms, turbidity, pumpage and temperature.

Table 4 shows the results of the study. With three clean basins in service, wash water usage decreased from 2.7 to 2.3 per cent; filter runs rose from 16.9 to 22.7 hr; removal of microorganisms increased from 84 to 88 per cent; and reduction of turbidity rose from 64 to 68 per cent. The amount of lime, ferrous sulfate, and chlorine used per 1 mil gal of water treated decreased. More aluminum sulfate was required, as it was applied to two-thirds of the water rather than half. The decreased cost of chemicals and wash water under these conditions is estimated at $63 per day. Applying the previously determined 60-day reduction in basin shutdown time to the estimated saving of $63 per day, the total annual saving in chemicals and wash water is $3,780. The elimination of hand labor—with mechanical

TABLE 3

*Sludge Quantities and Cleaning Labor Before and After Scraper Installation**

Item	1946	1952
Number of cleanings	3	2
Average period between cleanings—*days*	133	166
Total volume through basins†—*mil gal*	147,919	117,627
Total drained sludge—*cu yd*	73,331	18,395
Volume drained sludge per 1-mil gal throughput—*cu yd*	0.496	0.156
Total direct labor—*man-hr*	7,047	2,232
Labor per 1,000 mil gal—*man-hr*	47.6	19.0
Volume drained sludge per man-hour—*cu yd*	10.4	8.2

* Data from Mrva (*1*).
† Period covered does not correspond to calendar year.

TABLE 4

Average of 7-Day Periods During and Following Cleaning of Settling Basins, 1953–1955

Item	3 Basins	2 Basins
Average daily pumpage—*mgd*	335.2	337.7
Average number of microorganisms		
Raw water—*per ml*	1,000	923
Settled water—*per ml*	124	149
Percentage removal	88	84
Average turbidity—*ppm*		
Raw water	6.5	8.7
Settled water	2.1	3.1
Percentage reduction	68	64
Average filter runs—*hr*	22.7	16.9
Wash water usage—*per cent*	2.3	2.7
Chemicals applied—*lb/mil gal**		
Aluminum sulfate	61	48
Lime	23	29
Ferrous sulfate	28	44
Chlorine (oxidation)	3.7	5.8

* Total pumpage.

sludge removal from all basins—is estimated to save $3,800 annually. Thus the total annual saving for reduced wash water and chemicals and the elimination of hand labor for cleaning is $7,580.

To effect the foregoing reduction in labor, chemical, and wash water costs, sludge removal equipment must be installed and maintained. Assuming 8 per cent for fixed charges, maintenance, and operation, a capital expenditure of up to $95,000 would be justified for such an installation.

The existing sludge scrapers, which serve one-third of the basins, cost $210,000 installed. Current costs would be about $510,000 for the same equipment or approximately $1,000,000 for the equipment required to serve all of the basin area. If such equipment were installed, the net annual increase in operating expenses and fixed charges would be $72,420.*

Obviously, on the basis of this estimate alone, there is no justification for the installation of further sludge removal equipment. Before a final conclusion is reached, consideration must be given to possible improved operation resulting from continuous sludge removal. To analyze this, some study of what happens between present cleanings is required.

Operation Before Cleaning

Actually there is little evidence to indicate that plant operation is impaired as the result of sludge accumulating in the settling basins between

TABLE 5

*Monthly Operation Between Cleanings of Settling Basins, 1953–1955**

Item	Jun.	Jul.	Aug.	Sep.
Pumpage—*mgd*	384.1	396.8	390.1	360.4
Filter runs—*hr*	11.6	13.2	27.4	25.2
Microorganisms				
Raw water—*per ml*	1,518	948	415	485
Settled water—*per ml*	222	154	58	62
Percentage removal	85	84	86	87
Turbidity				
Raw water—*ppm*	3.7	4.0	3.7	4.0
Settled water—*ppm*	2.0	2.3	2.2	2.0
Percentage removal	46	42	40	50

* Figures show 3-year averages for months indicated.

* In a discussion of this paper from the floor, Oscar Gullans, Chief Water Chemical Engineer of the South District plant, stated that the present practice of starting removal of sludge immediately after drainage of the basin has reduced the shutdown time for cleaning below the average time shown in the paper. Direct hours of labor per unit of sludge have not been reduced. The estimated savings in chemicals and wash water, with complete mechanical cleaning, would be decreased for any reduction in cleaning time below the average time shown. The net effect of this reduction would be to decrease the savings resulting from installation of the mechanical scrapers.

cleanings. If such were the case it would be apparent in the efficiency of operation of three freshly cleaned basins as contrasted with the same basins just prior to the next cleaning. The 3-year averages (1953–1955) of the 7-day periods just after the three clean basins went into service and just before one basin was withdrawn for cleaning show somewhat better efficiencies for the clean basins when judged by the percentage removal of microorganisms and turbidity. The clean basins removed 88 per cent of the microorganisms, the dirty basins 86 per cent. The clean basins removed 68 per cent of the turbidtity, the dirty basins 65 per cent. Interestingly enough, filter runs were longer for the dirty basins but this resulted from lower microorganism counts in the raw water reaching the basins.

Additional and probably more substantial evidence is offered by average operating figures for the months of June, July, August, and September for the years 1953–1955. (Spring cleaning is completed in April or May, and fall cleaning begins in October.) Averages are shown in Table 5.

Several interesting facts are disclosed by this compilation. The longest filter run occurred in August, when the microorganism counts were the lowest. Percentage removal of both microorganisms and turbidity in the settling basins was better in September than in any one of the other 3 months, although this was the month immediately prior to the fall cleaning. The overriding effect of microorganisms in the settled water on length of filter runs is shown by the short filter runs in June and July when the average runs were 11.6 and 13.2 hr, respectively. It seems probable that the maintained efficiency in the settling basins over the summer months resulted from the depth of the basins. Even with sludge accumulations there still re-

TABLE 6

Relationship of Occurrence of Microorganisms in Raw Water to Average Water Temperatures, 1953–1955

Month	1953		1954		1955		3-year Avg	
	Microorganisms *per ml*	Temperature *°F*	Microorganisms *per ml*	Temperature *°F*	Microorganisms *per ml*	Temperature *°F*	Microorganisms *per ml*	Temperature *°F*
Jan.	1,113	33	696	33	1,445	33	1,085	33
Feb.	655	33	521	35	893	32	690	33
Mar.	646	34	675	35	1,139	34	820	34
Apr.	906	43	1,013	44	1,037	43	985	43
May	1,238	52	1,050	51	1,544	52	1,277	52
Jun.	1,935	61	1,131	55	1,488	54	1,518	57
Jul.	1,298	62	723	66	823	68	948	65
Aug.	470	70	283	71	493	72	415	71
Sep.	554	66	481	67	421	66	485	66
Oct.	1,110	60	992	56	799	56	967	57
Nov.	1,101	49	1,587	48	1,910	43	1,533	47
Dec.	846	39	1,949	36	2,214	32	1,670	36
Avg	989	50	925	50	1,184	49	1,033	50

mained a minimum of 11 ft of water above the highest sludge deposition.

The city of Chicago has under construction at the present time a 30-mil gal reservoir at the terminal of the Western Avenue tunnel and is taking bids on a new tunnel in 79th Street. These will improve the hydraulics of the South District system sufficiently to permit filter plant operation at a uniform daily rate with hourly variations in consumption absorbed in the filtered-water and storage reservoirs. These improvements, although designed for other reasons, should help settling-basin performance if the commonly accepted theory that basins should operate at a uniform rate is correct.

Microorganisms

Because microorganisms play a predominant part in filter plant operation at Chicago, supplemental information on their occurrence is included in Table 6. This table shows monthly averages of their occurrence in the raw water during the past 3 years. Also included as Table 7 is a study made several years ago of the relationship of turbidity and microorganisms in the settled water to the length of filter runs. These data are derived from the daily records for 2 years, or 730 observations, during which time the average pumpage was 308 mgd.

TABLE 7

*Relationship of Microorganisms and Turbidity in Settled Water to Average Length of Filter Runs, Jul. 1, 1948–Jun. 30, 1950**

Turbidity *ppm*	Microorganisms per ml:					
	0–49	50–99	100–149	150–199	200–249	More than 250
	Average Length of Filter Run—*hr*					
0–0.99	32.4	28.4	23.2	22.7	19.9†	21.6†
1.0–1.99	28.3	27.4	22.0	18.6	14.2	18.8
2.0–2.99	27.5	24.5	25.4	15.3†	13.5†	11.9†
3.0 and more	25.6†	27.6†	16.1†	8.8†	—	13.0†
Weighted Avg	29.5	27.1	22.6	18.9	15.8	18.2

* Average throughput, 308 mgd.
† Less than ten observations.

Summary

1. This study relates specifically to rectangular settling-basin operation in double-decked, parallel-flow structures with Lake Michigan water under loads in excess of nominal plant rating and at extremely variable rates.

2. Installation and operation of mechanical sludge removal equipment in one-third of the settling basin area reduced drained sediment, requiring hand cleaning, by 68 per cent.

3. Before installation of equipment, undrained-sludge depths in the upper section of the three double-decked, parallel-flow basins averaged 85 per cent of depths in the lower sections. Drained-sludge depths averaged 48 per cent.

4. After installation of equipment in the first third of the upper and lower sections of the basins the undrained-sludge depths in the unequipped upper sections averaged 83 per cent of depths in the similar lower sections. Drained-sludge depths averaged 62 per cent.

5. Shutdown time for cleaning basins can be reduced from 20 per cent of the year to less than 4 per cent by installing complete sludge removal equipment.

6. Net annual increase in operating expenses and fixed charges with complete sludge removal equipment is estimated at $72,420.

7. Plant operation is apparently not impaired by accumulation of sludge between cleanings.

8. Length of filter runs is largely dependent on the number of microorganisms in settled water. During months of maximum pumpage (June, July, August, and September 1953–1955) the average number of microorganisms in the settled water varied from 58 to 222, with the average length of filter runs varying from 11.6 to 27.4 hr. Average pumpage during these months varied from 120 to 124 per cent of nominal plant rating. The maximum day's pumpage was 153 per cent of nominal plant rating and the maximum hour was 165 per cent.

Source of Data

Statistical data used (other than from Mrva) are from daily logs and monthly and annual reports of the Department of Water and Sewers, James W. Jardine, Commissioner. The Bureau of Water—W. W. DeBerard, Chief Water Engineer, and H. H. Gerstein, Assistant Chief Water Engineer—through its Division of Water Purification, J. R. Baylis, Engineer, operates the South District Filtration Plant, with Oscar Gullans in immediate charge and J. C. Vaughn as his assistant.

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Evaluation of New Algicides for Water Supply Purposes

C. Mervin Palmer

A paper presented on May 7, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by C. Mervin Palmer, Research Biologist In Charge, Interference Organisms Studies, Water Supply & Water Pollution Program, Robert A. Taft San. Eng. Center, USPHS, Cincinnati, Ohio.

FOR the control of algae in water intended for domestic and industrial uses, only two basic chemicals—copper sulfate and chlorine—are commonly recommended (*1*, p. 110). This is in contrast to the rapidly increasing number of chemicals which are being made available for the control of various pests in commodities other than water. Many varieties of fungicides, bactericides, insecticides, and herbicides are being approved for use in fighting disease in humans, domestic animals, and agricultural crops, or in preventing loss of quality in foods, fibers, and many manufactured products.

The development of new algicides has lagged behind the discovery of other pesticides for several reasons. First, the price of water to the consumer is so low that only the application of an inexpensive algicide is economically possible. For example, actidione, an antibiotic substance isolated from the beers of streptomycin-producing organisms, appears to be very effective at a concentration of 2 ppm in preventing the growth of many of the green algae which are resistant to copper sulfate. At a cost of several dollars per gram, however, it is obviously not the answer for treatment of these algae in waters, although it may be very useful in the research laboratory in removing the green alga, *Chlorella,* from cultures of blue-green algae.

Secondly, the procedures for mass screening of algicides were not developed until recently, primarily because few of the algae were available as pure cultures, which are required for the establishment of reliable laboratory screening tests for new algicides (*2*). This condition is still true for a considerable number of the algae which cause most of the problems in the water supply. It is still impossible to grow such algae as *Synura, Dinobryon, Spirogyra, Asterionella, Coelosphaerium, Synedra,* and *Fragilaria* in pure cultures satisfactory for the screening tests.

The third reason why acceptable algicides are so limited is that a chemical that is nontoxic to humans as well as to fish is required. Although mercuric chloride and lead arsenate may be used to control organisms even on fruits which are eventually to be used for human consumption, they are not likely to be recommended for use in water supplies, in spite of the fact that they are effective in controlling algae.

Finally, in general practice, no distinction has been made between the algae which need to be controlled and

those which are causing no problems. Better records of the particular forms which are actually causing problems will allow the use of selective algicides in local situations where the offending algae may be concentrated, or, in a mixed algae population, in low concentrations just sufficient to control the algae for which the particular algicide is known to be effective.

Future Possibilities

The initial research on algicides to date has proceeded far enough to indicate promising possibilities for the future. Included among the groups of chemicals which show promise are the quaternary ammonium compounds, the rosin amines, the quinones, activated silver, urea compounds, chlorophenates, antibiotics, organic zinc, modified copper compounds, and modified chlorine compounds (*3*). Several commercial products, containing one or more of these chemicals, are already on the market and advertised for use in controlling algae in swimming pools, cooling systems, and certain other bodies of water.

Quaternary Ammonium Compounds

One of the quaternary ammonium compounds tested at the Sanitary Engineering Center was particularly effective against a number of the green algae. At 1 and 2 ppm it controlled more than twice as many kinds of green algae as did copper sulfate and at 0.5 ppm it controlled 29 per cent of green algae while the same concentration of copper sulfate controlled none. At this same concentration the quaternary ammonium compound was toxic to the filamentous green alga, *Stigeoclonium,* which is relatively resistant to algicides. If this compound should be found to be toxic to other filamentous forms, such as *Cladophora* and *Pithophora,* it would have potentialities for use in controlling some of the most troublesome of the attached algae. This chemical appeared to be less toxic to fish than was copper sulfate, its 48-hr median tolerance limit (TL_m) value being 0.65 ppm compared to 0.19 ppm for copper sulfate.

Rosin Amines

The rosin amine D sulfate has exhibited some selective toxicity for certain diatoms while the rosin amine D acetate was more general in its algicidal properties. The latter at 2 ppm controlled 90 per cent of all algae tested, compared to only 53 per cent controlled by the same concentration of copper sulfate. At 0.5 ppm the rosin amine controlled more than 3 times as many of the algae as did the copper sulfate. Fish toxicity tests indicated that in like concentrations of the active ingredient the rosin amine was possibly no more toxic to the fish than was copper sulfate. Like the copper sulfate, however, its effectiveness seems to vary in waters of different pH, hardness, and other factors.

Quinones

The quinones have been studied by Fitzgerald, Gerloff, and Skoog (*4*), who have demonstrated that 2,3-dichloronaphthoquinone is selectively lethal in exceedingly low concentrations to certain bloom-producing blue-green algae. Several compounds of the quinone type are toxic in concentrations of less than 1 ppm. Tests at the Sanitary Engineering Center indicated that 2,3-dichloronaphthoquinone was clearly the most selective of the algicides which have been tried. At 0.5

ppm it controlled 28 per cent of the blue-green algae under test without being toxic to any of the diatoms and green algae. Its lack of toxicity to green algae is striking. Even at 16 ppm it had little or no effect on the growth of 50 per cent of the green algae while controlling all of the blue-green algae tested. In alkaline water the chemical has a tendency to be modified so that it becomes less effective than it would be in neutral or acid waters.

Activated Silver

Colloidal or "activated" silver has been made available commercially and advertised as an algicide to be used at concentrations of up to 20 ppm of the gross material. Tests which were made with one sample, using 2 ppm on cultures of algae, showed no effect on green algae and diatoms. It did retard growth of some blue-green algae temporarily. Another sample of colloidal silver described as containing 33 per cent silver nitrate was tested at various dilutions on cultures of algae. The lowest concentration which prohibited the growth of all the cultures was 1.25 ppm of the gross material. When it was added to actively growing cultures of algae the amount which was required to destroy the algae was higher. The diatom, *Nitzschia,* for example, was prevented from developing in media containing 0.31 ppm, but an actively growing culture was not destroyed, even when the concentration of the colloidal silver material was raised to 5 ppm. Until it is possible to ascertain the amount of colloidal silver present in the substance, however, it is impossible to give accurate figures on the algicidal properties of the active ingredient. The tendency of the silver nitrate to cling to glass surfaces also makes it difficult to conduct reliable tests in the laboratory.

Urea Compound

The urea compound, 3-(*p*-chlorophenol)-1,1-dimethylurea, commonly known as CMU, has a reputation for being toxic to practically all plants and relatively nontoxic to animals. Tests on algae indicate that it is an effective general algicide at concentrations of a few parts per million. If it is used, however, there is the danger that it will also destroy trees, shrubs, grass, and other plants with which it may come into contact through seepage or drainage into the surrounding soil.

Chlorophenate

A few years ago sodium pentachlorophenate was described as a chemical particularly toxic to algae, but unsafe for treatment of domestic water supplies or for swimming pools (*5*). Tests on cultures of algae, however, have indicated that a concentration of 1.5 ppm only temporarily retarded the growth of several algae and had no effect on the green alga, *Chlorella.* It is considered to be stable in alkaline waters and not subject to reduction of its effectiveness in water of high pH.

Antibiotics

It was found rather unexpectedly that streptomycin, neomycin, terramycin, and certain other antibiotics were particularly effective against the blue-green algae (*6*) while actidione, referred to earlier, was selectively toxic to certain green algae and diatoms. In the case of streptomycin, the growth of 100 per cent of all the cultures of blue-green algae was inhibited by 1 μg/ml equivalent of pure streptomycin base.

This concentration inhibited only 18 per cent of the green algae. A concentration as low as 0.015 μg/ml retarded the growth of 25 per cent of the blue-green algae. It is conceivable that the natural production of antibiotics by aquatic streptomycetes and other organisms in the water may serve to modify drastically the growth of certain algae in water supplies. Some day enough may be known about the interactions of organisms in water to encourage the desirable forms in their battles with the undesirables.

Organic Zinc

The organic zinc compound, zinc dimethyl dithiocarbamate (ZDD), has proved to be one of the most effective general algicides among those tested. It inhibited the growth of the blue-green alga, *Microcystis*, even at the lowest concentration used, 0.004 ppm. In addition, at a concentration of only 0.25 ppm, it controlled all of the diatoms tested, together with 43 per cent of the blue-green algae and 18 per cent of the green algae. This made a total of 40 per cent of all algae tested, while five other promising algicides—including copper sulfate—controlled 0–20 per cent, at the same concentration. The one great drawback to ZDD, however, is its high toxicity to fish. In the Sanitary Engineering Center tests, the 96-hr TL_m value for this compound was 0.008 ppm, while for copper sulfate it was 0.18. This high toxicity to fish would obviously limit its usefulness as an algicide except in closed systems, or possibly in isolated cases as a selective algicide in very low concentrations. The official Food and Drug Administration tolerance for ZDD for fruits and vegetables is 7 ppm, which is similar to that listed for DDT, toxaphene, and lead arsenate (*7*). The closely related compound, ferric dimethyldithiocarbamate, is also algicidal, but other investigators have found that it may cause damage to fish tissues when used at 0.5 ppm (*8*). The screening of a large number of compounds closely related to ZDD might uncover a safe and effective new algicide.

Copper and Chlorine Compounds

Copper sulfate is much less effective against algae in hard water than in soft water because it combines with calcium carbonate to form a precipitate of copper carbonate. For this reason, copper-citrate combinations, and others which tend to allow the copper to remain in solution in hard water, have been recommended. Often one of their limitations is the relatively high cost.

Copper and chlorine, when used together, are frequently more effective than when either is used alone. Cuprichloramine, a combination of copper, chlorine, and ammonia, has also been employed (*1*, p. 122; *9*). More recently, chlorine dioxide has been tested and found to show promise as a general algicide (*10*). It is claimed to have the added advantage of reducing some of the tastes and odors produced by the algae. The cost of treatment is estimated to be much higher than that of copper sulfate or chlorine.

Summary

The screening tests for algicides thus far conducted serve to indicate that there are a number of chemical groups which contain compounds that are algicidal. The promising chemical groups include the inorganic salts, organic salts, rosin amine compounds, antibiotics, quinones, substituted hydrocarbons, quaternary ammonium compounds, amide derivatives, and

phenols. The algicides satisfactory for use in domestic water supplies will have to be not only economical to apply but nontoxic to animal life and to plants other than algae. As these will probably cost more per pound than the algicides now in use, they will probably be used only where careful plankton records are kept, which would permit early localized treatment where records show that undesirable species of algae are threatening to increase in numbers. Algicides which are selectively toxic to the algae producing tastes and odors, the filter-clogging algae, the bloom or slime producers, and other interference types, would be particularly valuable. A promising start has already been made in this direction but there are as yet no completely new algicides which would appear to be safe to recommend at present for use in public water supplies.

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Factors Affecting Filtration Rates

Herbert E. Hudson Jr.

A contribution to the Journal by Herbert E. Hudson Jr., Cons. Engr., Hazen & Sawyer, Engrs., New York, N.Y. The paper was prepared as a progress report for the Subcommittee on Water Supply of the Committee on San. Eng. & Environment, National Academy of Sciences, National Research Council, Washington, D.C., and was approved by the committee on Jun. 5, 1956.

A STRONG trend toward the use of filtration rates higher than the customary 2 gpm per square foot is apparent in current rapid sand filtration design and operation. There is a great deal of evidence to support the desirability of this trend. Frequently, however, practice is following the trend quite indiscriminately.

A sound decision on the rate of filtration for a given water purification plant requires consideration of [1] the desired effluent quality, [2] the character of the applied water, [3] the size of the sand to be used, [4] the depth of the filter bed, and [5] the condition of the filter bed.

Desired Effluent Quality

The water leaving a purification plant must at all times be free of pathogenic organisms. In the early days of slow sand filtration the filter was the sole guardian of water quality. With the development of coagulation, sedimentation, and prechlorination, followed by rapid sand filtration and postchlorination, this situation changed greatly.

A modern rapid sand filtration plant contains a defense in depth against the passage of pathogenic organisms. The first line of defense is sedimentation (which is aided by chemical treatments such as coagulation and softening). Usually the main line of resistance is chlorination, which is expected to destroy completely all pathogenic organisms. The final defense, available if there is penetration through or failure of the main line of resistance, is filtration. Thus modern water purification contains several defenses against the threat of disease-producing organisms. Because of this, filtration has come to be regarded as simply a polishing treatment that usually carries only a minor share of the burden of protecting the public health.

The defenses against waterborne disease have their weak spots, caused by faulty design (as a result, usually, of a lack of information), by mechanical failures, and by human failures on the part of those who operate the treatment works. Because of these weaknesses, complete reliance should not be placed on pretreatment and chlorination when there is any substantial likelihood of attack by disease-producing organisms. In addition, filtration is still the principal bulwark against motile chlorine-resistant pathogens.

Sound public health practice therefore requires that the filtration process be capable of high effectiveness. Ex-

perience has taught that rapid sand filters having 2 ft of 0.5 mm sand, operated to 8-ft loss of head at filtration rates of 2 gpm per square foot of filter area, usually provide safe water. The margin of safety provided by filtration under these conditions is unknown, however, and any liberalization of practice that may have an adverse effect on filtered-water quality must be considered carefully.

Fortunately, simultaneous failures of both filtration and chlorination are rare. It is therefore difficult to obtain bacterial data on filtration failures, and some other determination must be used to evaluate them. Since the object of filtration is to remove suspended material—including pathogenic organisms—from the water, it is reasonable to use determinations of the suspended matter in the filtered water as indexes of filtration effectiveness. There are several satisfactory methods for measuring suspended material in filtered water.

In a recent paper, Baylis (*1*) summarized his experience in measuring the quality of filtered water. He pointed out that, owing to prechlorination, the bacterial quality of the filtered water was uniformly excellent in trials of high-rate filtration at the Chicago South District Filtration Plant. He described results obtained with the Baylis turbidimeter, which measures colloidal turbidity to 0.1 unit, and with the floc turbidimeter. Baylis did not report on the "St. Louis" turbidimeter, which appears capable of measurement of turbidity to 0.1 unit. All these devices rely on standards of turbidity which are laborious to prepare and use.

Baylis found the cotton plug filter the most sensitive device for determining the amount of coagulated matter passing through the filters, and recommended its use generally for control of rapid sand filtration processes. Averages of results indicate that the device is sensitive to 0.001 ppm. The passage of coagulated matter is most objectionable. It indicates unsatisfactory filter performance and leads to the accumulation in mains of sludge deposits that are subsequently scoured back into suspension in times of high demand.

Using turbidity measurement equipment of the type described, the Chicago staff has striven to produce filtered water completely free of floc and having a turbidity of less than 0.1 unit, and has succeeded in maintaining annual average turbidity, as determined by cotton plug filters, below 0.087 unit for filters operated at rates of 5 gpm per square foot, with values as low as 0.028 unit for filters at 2 gpm per square foot. The principal tools used for the attainment of this result were a lengthy flocculation period, several hours of settling, filters that were kept scrupulously clean, close control of coagulant dosage, strengthening of coagulation with acid-treated sodium silicate during periods of weak flocculation, and 'round-the-clock supervision by professionally trained engineers and chemical technicians on all shifts. When such control and preparatory treatment are available, high filtration rates are safe to use.

The requirement of filtered-water turbidity below 0.1 unit, with complete freedom from floc turbidity, seems unreasonably high to some water works men, who contend that a lower standard of filter performance can be accepted if reliance is placed on chlorination to complete the defense of health. This contention overlooks the occa-

sional failures of chlorination which do occur as a result of mechanical troubles and human error or as a result of sudden changes in pollution loadings.

It is also contended that such a standard of filtered-water quality is extreme, and difficult to meet. That this standard is being met with filtration rates of 5 gpm per square foot and an effective size of sand of 0.65 mm demonstrates that it should be widely attainable with lower filtration rates and finer sand.

This standard of quality is much more rigorous than the existing federal standard for drinking water. It is suggested that, in light of the capabilities of modern filtration plants, the federal standards could probably be made much more stringent for application to filtered-water supplies. The subject deserves future study.

Character of Applied Water

In modern water treatment, settled-water turbidities of about 1 ppm are sometimes attained through good flocculation and well designed sedimentation. Even in plants that attain such results, however, tenfold increases in turbidity (and bacterial loading) appear at times in the settled water as a result of changes in raw-water quality or mishaps in dosage. The principal difficulties in producing high-quality filtered water appear to result from changes in the concentration and nature of the settled-water turbidity. These may be associated with changes in the mineral quality of the water.

In the choice of parameters for filter design, what matters most are the critical filterability conditions. The two critical conditions lie at the extremes of filterability. When flocculation is strong and the suspended matter is such that penetration of coagulated matter into the bed is small (as when high concentrations of filamentous microorganisms serve as reinforcement), the limiting condition is a high clogging rate. In this situation, high filtration rates result in very frequent backwashing. There is ordinarily no problem of suspended material in the filtered water under this condition. In public water supply work, this condition most commonly arises when demands for water are high, as in late spring, summer, and early fall.

On the other hand, when floc is weak—as when temperatures approach the freezing level and suspended material in the raw water consists largely of colloidal particles—the critical problem is the penetration of suspended material through the filter bed. Except for cases in which unsatisfactory dosage control or contamination causes weak flocculation, this condition is most likely to occur in cold water—during the winter months—at a time when demands on the plant are small. It is this condition with which the present study is especially concerned.

When flocculation is weak, poor sedimentation frequently accompanies poor filterability. Baylis observed (*2*) that great floc penetration seemed to have been accompanied by high settled-water turbidity. High concentrations of applied turbidity should be expected to produce early breakthroughs of flocculated turbidity during periods of weak flocculation.

Most experimental studies of relations between water quality and filtration rates have ignored the seasonal variance in floc strength. Nearly all these studies have been carried on under laboratory conditions or under

average plant conditions, so that the results obtained failed to provide data on the limiting period of weak flocculation.

Previous Filterability Studies

An empirical parameter for indicating floc strength sheds some light on the problem of filterability (*3*). This parameter was called the "floc strength index" and was expressed as $\frac{hd^3}{L}$ in which h was the loss of pressure through a bed thickness L at the time when measurable increase in the suspended matter in the effluent began to occur. The term d represents the effective size of the bed particles, expressed in millimeters. This parameter did not take rate of filtration into account as no data on filtered-water quality at various rates were available at the time it was developed.

The floc strength index was designed primarily to define the structural ability of floc particles to resist the forces occurring in the filter bed. A high value is associated with low penetration of floc into the bed and high clogging rates; a low value, conversely. The author has observed values of $\frac{hd^3}{L}$ ranging from greater than 25 to to less than 0.2. With 2-gpm filtration rates, 0.5-mm sand, 2-ft thick beds, and applied concentrations of turbidity in the range of 0.5–5 units, these values produced filter runs to 8-ft loss of head ranging from 1 to 150 hr. As an approximation:

$$T = \frac{25}{\frac{hd^3}{L}}$$

in which T is the length of run (in hours) for a filtration rate of 2 gpm per square foot.

Such a parameter may find use in connection with evaluation of pretreatment facilities, as suggested by AWWA committee E5.B2 (*4*). Values of $\frac{hd^3}{L}$ calculated from available data gathered during various researches on floc penetration (*5–8*) are given in Table 1.

From these data it will be seen that in the experiments on floc penetration for which data are available, the values

TABLE 1

*Floc Strength Indexes for Various Rapid Sand Filter Studies**

Source	$\frac{hd^3}{L}$
Harvard (*5*)	10.0
Baltimore (*6*)	3.4
Toronto (*6*)	2.2
Chicago (*7*)	
Uniform sand data	2.0
Most extreme value	0.2
Johns Hopkins (*8*)	†

* Filtration rates of 2 gpm per square foot and 0.5-mm sand were used.

† No breakthroughs occurred during pertinent runs on the filters studied at Johns Hopkins, so that floc strength index could not be determined. The value was in excess of 0.2.

of $\frac{hd^3}{L}$ exceeded 2.0, except in the Chicago measurements, collected during a period of weak coagulation which persisted for several weeks. Design cannot be based on relationships developed for noncritical conditions. On the other hand, it seems likely that values of $\frac{hd^3}{L}$ below 0.1 will occur very rarely and pose values too extreme for practicable design.

The floc strength index provided a basis for design of the filter so far as effective size of sand, terminal loss of

head, and depth of bed are concerned. In general, a similar basis for design was reached in the studies conducted at Harvard by Stanley (*9*), who also included the effect of rate of filtration

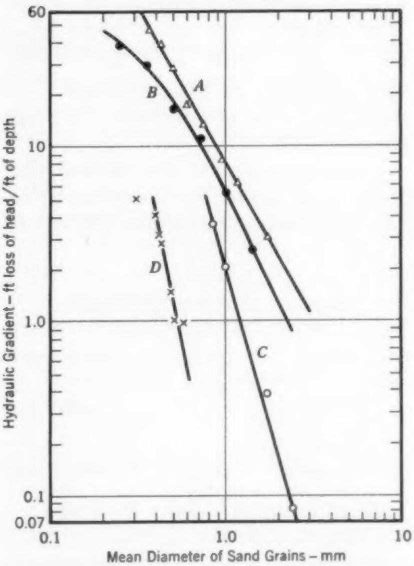

Fig. 1. Hydraulic Gradient at Which Sediment Penetrates Sand of Various Effective Sizes

Curves shown represent data from following studies: A, *Baltimore;* B, *Toronto;* C, *Chicago (uniform sands);* D, *Chicago (nonuniform sands). Data are for weak flocculation conditions (6, 7).*

in arriving at a relation which may be written as:

$$K_1 = \frac{hd^{2.56}V^{1.56}}{L}$$

in which V is the rate of flow, and K_1 is a constant. Inasmuch as Stanley's work was done with water that filtered clear with comparative ease, it is felt that this relationship may not describe the most critical condition.

Filter Bed Condition

Good results in clarifying water cannot be expected from fouled filters. The surface wash and high-velocity backwashes have come into general acceptance as devices for keeping the filter sand clean. Means for keeping sand clean are considered to be requisites in new plants, and are rapidly being added to old ones. It is hopeless to attempt quantitative analysis of the results to be obtained from filters whose sand beds crack, mound up, or pull away from the side walls. They cannot be relied on to produce clear water at all times.

Types of Filtration Systems

Brownell (*10*) has written a general discussion of filtration processes in which he makes an important distinction between two general types of filtration:

The filter medium is the essential part of a filter which retains the suspended solids but permits the flow of the fluid. In general, filter mediums may be divided into two groups: *en masse* mediums in which the medium is the primary filtering agent throughout filtration, as in the case of the sand in the sand filter; and initiating mediums in which a thin cloth or screen starts the formation of a filter cake which then becomes the filter medium. *En masse* filter mediums may be classified into two groups: crystalline or granular mediums that are cleansed by backwashing for reuse; and fibrous filter-mass mediums that must be replaced after collecting a quantity of solids (some may be reused after a special wash treatment). Initiating mediums may be divided into metallic and nonmetallic screens or cloths.

Sand is the most widely used *en masse* granular filter medium, largely because of

its availability, stability, and low initial cost. The sand forms a labyrinth path for the turbid fluid. Although a suspended particle may penetrate some distance into the bed by passing along the large channels, the particle will eventually be trapped in one of the smaller interstices, if the bed is of sufficient depth. A filter bed of fine sand may be replaced by a deeper bed of coarser sand to give the same filtration. The coarser sand will have the greater filter capacity because of its greater permeability.

A second important classification of filters divides them into two types: constant pressure and constant rate.

A complete review of filtration theory has been made by Heertjes (*11*). This review, which includes a bibliography of publications on filtration, sheds no light on constant-rate filtration, nor does it give information on the *en masse* problems which occur as sediment penetrates into the filter bed and through it. Much work has been done in the field of analysis of constant-pressure filtration (*12*). The extensive published analyses of constant-pressure and filter-cake filtration relationships seem inapplicable to the study of the rapid sand filter using constant-rate operation.

Constant-Rate Filtration Data

In 1936, the Committee on Filtering Materials for Water and Sewage Works of the Sanitary Engineering Division, ASCE, presented a report which summarized extensive work by L. F. Allen at Toronto and by James W. Armstrong at Baltimore (*6*). It appears that this report contained the earliest published information on critical depths of sand filters. The critical depth was defined as the maximum depth to which sediment will penetrate into a bed of uniformly graded sand operated to a loss of head of 8 ft of water when the filter is delivering a clear effluent at a rate of 2 gpm per square foot. The committee determined this critical depth by measurement of the solids trapped in the filter bed as determined turbidimetrically in successive layers of the bed. Allen

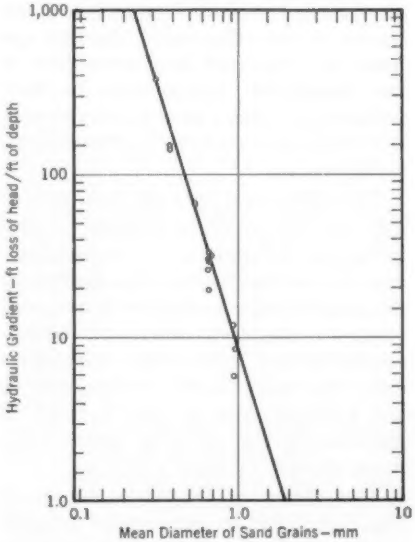

Fig. 2. Head Loss Increase per Unit Floc Penetration for Sand of Various Sizes

Data plotted are from Stanley's Harvard studies (5).

(*13*) called attention to the fact that the critical depth needs to be considered in the light of the amount of turbidity that appears in the effluent of the filter rather than in terms of the amount caught in the filter bed.

Accordingly, question may be raised about critical-depth determinations obtained solely on a basis of the measurement of suspended matter trapped

within the bed, particularly when the standard of clarity of the effluent is extremely exacting in comparison with the precision of the method used for measuring the solids gathered within the bed.

Figure 1 is a plotting of data gathered by Allen and Armstrong and the data used by the author in an analysis of filtering functioning (*3*). The author used data based entirely on the clarity of the effluents of the Chicago filters to determine the penetration of the suspended matter into the bed. Effluent turbidities were measured with the Baylis floc detector and the Baylis turbidimeter.

The differences between these curves may be the result of several factors, including differences in methodology for the determination, the nature of the coagulated material in water, and other considerations which will be discussed below. Allen's data are apparently the more nearly comparable to the Chicago data in the method of determining the point at which sediment passed through the filter.

The slopes of the lines drawn in Fig. 1 indicate an exponential relation between the hydraulic gradient at which sediment began to pass through the bed and the size of sand in the bed. For the Chicago data the values of the exponent are of the order of 3.0. For the Baltimore data, the value for the three coarsest filters is approximately 2.16, and for the Toronto data as a whole the slope is 1.73.

An intensive study of floc penetration into sand filters was made by Stanley (*5*). Owing to a change of experimental procedure during the sequence of observations, the data obtained are not all comparable. The data from Stanley's run No. 7, which was conducted at 2 gpm per square

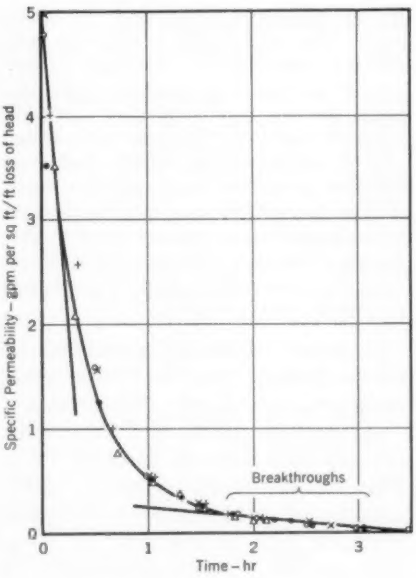

Fig. 3. Rate of Clogging, Diatomite Filters

Data are from Babbitt and Baumann run No. 42 (19). Symbols indicate following filtration rates (in gallons per minute per square foot): ○, *1;* ×, *2;* ●, *3;* +, *4;* △, *5.3.*

foot to determine effect of sand size on floc penetration, are shown in Fig. 2. The equation of the curve fitting the Chicago data in Fig. 1 and Stanley's data in Fig. 2 is:

$$\frac{hd^3}{L} = k$$

Fair (*14*) has pointed out that use of critical depth data in design of filters might well be by a size summation method such as that proposed by Fair and Hatch (*15*) and by Allen (*13*).

A paper by the author (*7*) dealt with the behavior of various types and

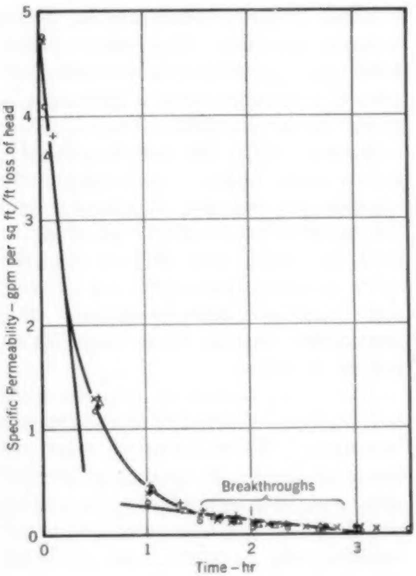

Fig. 4. Rate of Clogging, Diatomite Filters

Data are from Babbitt and Baumann run No. 43 (19). Symbols indicate same rates as in Fig. 3.

sizes of filtering materials in the passage of sediment through the filter beds. This paper did not contain usable information on the effects of rate of filtration, although data for one series of filters operated at 4 gpm per square foot were included. The data indicated that, when the filter runs were caused to vary by use of different filtering materials, the filtered-water quality was a function of the length of filter run.

Effects of Filtration Rates

In an investigation carried on under contract with the Engineer Research and Development Laboratories, Geyer and Machis (*8*) at Johns Hopkins University studied the penetration of solids into filtering materials under several filtration rates. As described in their report, they found that:

> A sand having a geometric mean size of 0.27 mm was treated at rates of 2.0, 3.6, and 6.0 gpm per square foot. For a terminal head loss of 60 cm of mercury (11.6 psi), the intermediate rate produced 1.75 times . . . and the high rate 2.65 times the volume of water produced by the low rate. This type of variation has been observed in earlier tests. However, there are exceptions, as indicated by data obtained in this test for 1.1-mm sand. In this case, there was a decrease of 6 per cent in the total volume of output at a terminal head of 60 cm of mercury.
>
> A study of the bacterial quality of the effluent of similar filters operating at different rates indicated that bacterial removals were about the same when compared on the basis of equal volumes of output. A comparison of the turbidity of the effluent of each of these filters appears to indicate a slightly higher turbidity in the effluent from the high rate filters. The turbidity of all the effluents remained below 0.3 ppm. . . .
>
> A study of the total quantity of solids removed by similar filters operated under different rates shows an appreciable increase in solids removal by the high-rate filters as compared with the low-rate filters. This observation is in conformity with similar observations made in previous tests. Reduction of the value of the total solids removal in the sand to a total solids removal per unit volume of water filtered indicates that the high-rate filters remove somewhat less of the solids present in the water. . . .

In general, the higher rates of filtration tend to drive the solids more deeply into the filter bed. This results in the production of more water with the same size of filter bed at the same head loss. If an effluent of equal quality is desired, however, a

deeper filter bed is necessary. Filter rates up to 4 and, in some cases, even 10 gpm per square foot are possible without any deterioration in quality of effluent, provided the proper grain size and depth of filter medium are selected.

In a study of relationships between filter performance and filter rates, Baylis (*17*) reported that runs were found to be inversely proportional to the 1.5 power of the rate of filtration. The clogging rate of the filter was also found to be inversely proportional to approximately the 1.5 power of the filtration rate. Baylis states:

> One probable reason why it is difficult to obtain more uniform data on the effect of the rate on the length of the run is due to the flocculated matter penetrating to different depths of the filter bed. It appears that some of the water is gradually squeezed out of the coagulated matter after it deposits in or at the surface of the sand bed, and that time is required for this to take place. When a large amount of coagulated matter goes to the filter within a certain time, due either to high concentration in the water or a more rapid flow of the water, there is not time for the coagulated matter to be compacted to its maximum density.

Baylis has recently made available the results of high-rate operation of full-size filters in the Chicago filtration plant (*1, 18*). These results indicate that the quality of the filtered water suffers in direct proportion to the filtration rate. This took place despite the close control over applied water quality exercised in the Chicago plant.

Baylis noted, however, that the high-rate filters appeared to produce disproportionately larger quantities of water per unit increase in head loss than those operated at standard rates. Geyer and Machis also observed this.

Filter runs are shortened by filtration rate increases. In instances where short runs constitute a problem, improved pretreatment is a prerequisite to increasing filtration rates.

Stanley's data for run No. 8 (*5*), which were gathered to evaluate the relation between rate of filtration and floc penetration, confirm the observations of Geyer and Machis and the more recent observations by Baylis, that disproportionately greater floc penetration results from increases in rate of filtration.

Sediment penetration into the filter bed increases when filtration rates are increased. When filtration rates are increased, however, quality of filtered water can be safeguarded by increasing the depth of the filter medium or by lowering the terminal loss of head. The remainder of the present study is devoted to examination of the available data in an attempt to evaluate quantitatively the effects of rate of filtration on floc penetration, especially under the most extreme conditions.

Role of Turbulence

In the foregoing discussion it has been assumed that the flow through the filter bed is laminar. There are, however, reasons to believe that turbulence may occur in filters at high filtration rates, with coarse sand, or at high terminal losses of head. If this is true, the relationship of filtration rate to floc penetration may be complex.

One method of approach (*3*), based on laminar flow, indicates that graphs of time against the reciprocal of head should yield straight lines.* This was

* The paper referred to (*3*) contained an erroneous definition. On page 868, the term *d* should be the weight of suspended matter per unit volume, as trapped in the filter, rather than "the density of the suspended matter in water," as given.

tested using data for various rates of filtration gathered on diatomite filters by Babbitt and Bauman (*19*) and sand filters by Geyer and Machis (*16*). In the process of testing this relationship it was found that excellent correlations of "specific permeability" against time were obtained. Specific permeability is the ratio of the rate of flow to the hydraulic gradient through the filter bed. These data are shown in Fig. 3–5. Insufficient data on head loss were reported in Stanley's measurements to justify trying this correlation with them.

Figures 3–5 indicate straight-line relations at low head losses; however, marked inflections in the lines occur and a second straight-line portion of each curve occurs. It is interesting to note that the data for the various filtration rates are nicely unified in the correlations. These data are for examples in which there was substantial penetration of floc into the filter beds. Head losses ran to more than 30 ft of water. Breakthroughs were recorded for the diatomite units, none for the sand filters.

The unexpected inflections in the curves indicated the existence of forces not considered in the derivation being tested and suggested the possibility that turbulence might play a part.

The clogging of an *en masse* filter proceeds from the surface first reached by the applied liquid toward the exit of the filter (*17, 20*). When the applied suspended matter is firmly bonded together by good coagulation, there may be relatively little penetration of suspended matter into the bed. When the suspended particles are not strongly bonded into groups of substantial size, the suspended matter penetrates into the bed. When the coagulant bond is weak (the condition called "weak flocculation"), suspended matter may penetrate entirely through the bed. In rapid sand filter operation, when the filter runs are 10 hr or more in length, unless the applied water contains very little suspended matter, there is substantial penetration of suspended matter into the bed.

Baylis (*17*) has described the process of filtration as one in which the greatest part of the applied water enters the filter bed through a very small proportion of the pores. These pores doubtless terminate or branch a short distance within the bed, and sediment-bearing water may be expected to move laterally to other nearby larger pores. Thus it is seen that the greatest part of the flow in a partially clogged bed takes place in only a fraction of the total pore space. This is a wholly different flow pattern than takes place in a clean sand bed, in which nearly all the pore space is available for flow. Even here, however, there is doubt as to whether all the pores are used by flow of clear fluids (*21*). It therefore appears that velocities within the constant-rate sand filter increase as the clogging proceeds, possibly to such a magnitude that turbulence may occur.

It has been widely recognized that, as the Reynolds number increases beyond a certain value, resistance to flow through beds of granular material exceeds that given by Darcy's law. The value at which the departure from the Darcy law takes place has been subject to several different determinations. There are also differences of opinion as to whether this departure involves turbulence or may instead be a "transitional zone." It seems to the author that the beginning of the departure is due to the commencement of eddying resistance or turbulence in a few of the pores. The pores are not all of

the same size and, as velocity increases, the eddying resistance probably commences first in the largest ones. Full turbulence in beds of uniform material apparently does not occur until the Reynolds number has increased by 10^4 greater than the value at which turbulence first exerts its influence *(22)*. It seems reasonable to assume that the greatest share of the filtration process would make use of the larger pores, and thus be profoundly affected by turbulence in them.

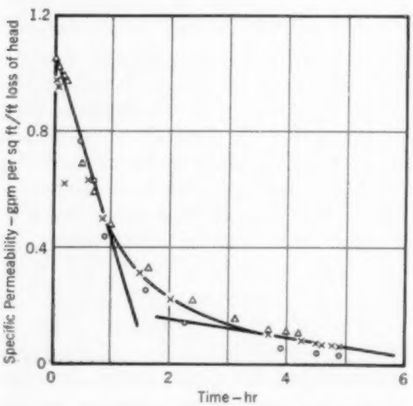

Fig. 5. Rate of Clogging, Sand Filters

Data are from Geyer and Machis run No. 8 (16). Symbols indicate following filtration rates (gallons per minute per square foot): ○, 2; ×, 3.6; △, 6.

In order to estimate the extent to which turbulence or eddying resistance may be involved, an effort was made to evaluate the critical velocity, above which some of the pores in the sand bed produce friction losses exceeding those of the Poisseuille-Hagen relation *(20)*. Use was made of data from uniform lead shot and rounded sands *(22)*. These data were plotted in a Lindquist diagram *(23)*, Fig. 6, from which it was determined that the critical value lies at a Reynolds number smaller than 2, when the Reynolds number is computed in consistent units as the product of approach velocity (V), particle diameter (d), and fluid density (ρ), divided by viscosity (μ).* Fair estimates the critical value from data on settling rates of sand at 0.5 *(24)*.

In a filter containing clean sand of 0.5-mm diameter, operated at 2 gpm per square foot, and at a temperature of 0°C, the Reynolds number, $\frac{Vd\rho}{\mu}$ is about 0.4. Under these conditions, a 24-in. deep bed will have a loss of head of about 1.2 ft.

It therefore appears that a large part of the work of the rapid sand filter is done in or beyond the transitional range, although much of it may involve turbulence to a minor extent.

Sediment transport and dislodgment are considerably greater under turbulent flow than under laminar conditions. It may therefore be anticipated that there would be greater movement of sediment into the bed at higher filtration rates. Thus, greater penetration and passage of sediment through the bed should be expected as a result of increased turbulence caused by use of higher filtration rates.

The available data on the initiation of turbulence lead to the supposition that turbulence might be responsible for the inflections in the curves in Fig. 3–5.

* T. R. Camp has suggested that the porosity factor be included in the expression of Reynolds number (private communication). This would improve the value of the term, but complicates the problem. In addition, there is as yet no agreement on how to take porosity into account. Figure 6 shows it to be a very important factor.

If complete turbulence played a major role in the process of rapid sand filtration, the pore velocity might be expected to increase as the square root of the head loss increases. For a clean filter, pore velocity is proportional to the approach velocity, V_0. In the clogged portion of the bed, after time t, the pore velocity V_t would be proportional to the ratio of the square roots of the initial and the clogged losses of head:

$$\frac{V_t}{V_0} = \frac{\sqrt{h_t}}{\sqrt{h_0}}$$

in which h_t is the loss of head through the clogged portion of the bed, and h_0 is the loss of head through the same length of clean bed. It is well established *(25)* that:

$$h_0 = c\frac{1}{d^2}$$

in which c is a constant of proportionality. The value of h_t depends upon the length of time the filter is operated, the concentration of suspended matter in the applied water, and so forth. Therefore:

$$V_t = V_0\frac{\sqrt{h_t}}{\sqrt{c \div d^2}}$$

It follows that, for the state of flow through the clogged portion of the bed:

$$\mathbf{R} = \frac{\left(V_0\dfrac{\sqrt{h_t}}{\sqrt{c \div d^2}}\right)d\rho}{\mu} = \frac{\rho}{\sqrt{c}\mu}\,(V_0\sqrt{h_t}d^2)\,..\,(1)$$

in which μ is the viscosity, ρ is the density of the fluid, V_0 is the approach velocity, and h_t is the loss of head after any stated time, assuming turbulence. **R**, of course, is the Reynolds number.

All the available data were gathered under uniform temperature conditions, so that viscosity was constant and may be ignored. The penetration data gathered by Stanley have been plotted against a modified Reynolds number in Fig. 7 and 8. Only data from Stanley's runs No. 7 and 8 were used,

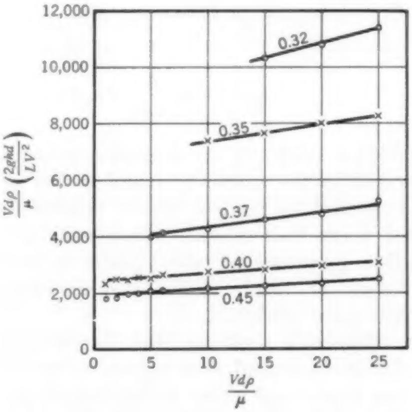

Fig. 6. Departure From Streamline Flow, Rounded Uniform Sands of Various Void Ratios

for his data indicate that the earlier runs were not comparable and do not appear to be applicable for this study. V was expressed in gallons per minute per square foot, d in millimeters, and h in feet of water.

Figures 7 and 8 indicate a relationship somewhat like that shown in Fig. 6. The assumption that turbulence plays a role in floc penetration yielded an excellent correlation under the conditions met in Stanley's experiments.

Baylis has pointed out that, under conditions of very weak flocculation, effluents become turbid at low head losses. Should this happen at heads so low as to be entirely in or very near the laminar flow range, we should expect, by a similar process of reasoning:

$$\frac{V_t}{V_0} = \frac{h_t}{h_0}$$

$$V_t = V_0 \frac{h_t}{c \div d^2}$$

$$\mathbf{R} = \frac{\left(V_0 \dfrac{h_t}{c \div d^2}\right) d\rho}{\mu}$$

$$= \frac{\rho}{c\mu}\,(V h_t d^3) \quad (2)$$

This is evidently the condition encountered by the author *(3)* and it may be expected to control under conditions of weak flocculation, where considerable penetration takes place at low losses of head, and correspondingly low pore velocities.

Stanley's data on floc penetration for various sand sizes appear to correlate fairly well under either hypothesis, but Eq 1 above reconciles his data on various filtration rates better than does Eq 2. It is therefore believed that his data were gathered under conditions in which turbulence controlled.

In either event, it seems clear that penetration of turbidity into or through the filter should be expected to increase with the rate of filtration, apparently in direct proportion.

The present analysis of the problem of behavior of sand filters under transitional or turbulent conditions is exploratory. Further study of this relationship would be valuable, but would require more data than are now available.

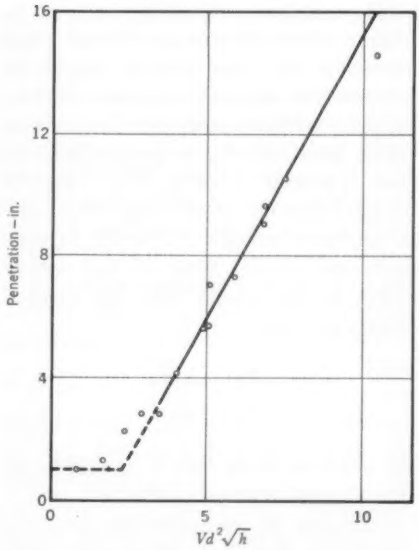

Fig. 7. Relation Between Floc Penetration and Reynolds Number, Sand Filters

Data are from Stanley's run No. 7 (5). Size of sand particles varied. Filtration rate was 2 gpm per square foot.

The most critical condition, of ready penetration of floc through filters at relatively low heads, seems to be governed by Eq 2.

It is interesting to note that it follows from Eq 2 that constant-pressure filtration might yield better quality water than constant-rate filtration. Possibly Jackson had this in mind in conducting the recently-published studies at the Dalecarlia plant at Washington, D.C. *(26)*, in which gradually varied flow rates were used.

Summary of Findings

The laminar-flow relation appears to govern the design of filters to meet critical weak flocculation conditions. Under these circumstances, it appears

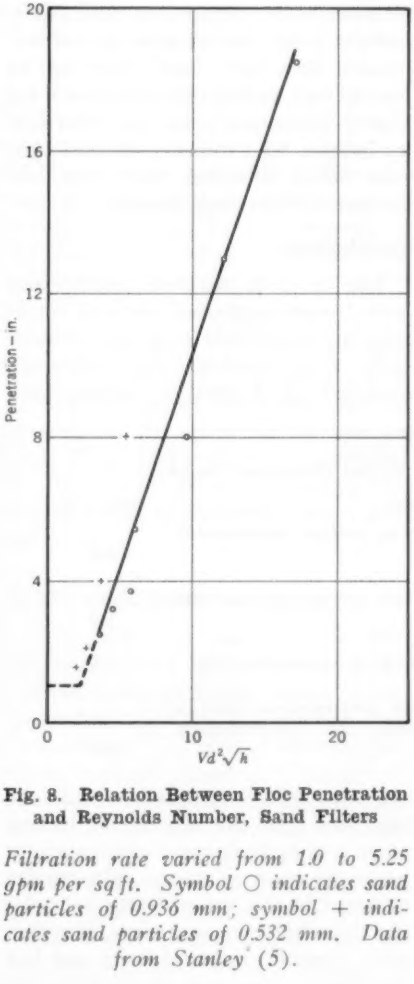

Fig. 8. Relation Between Floc Penetration and Reynolds Number, Sand Filters

Filtration rate varied from 1.0 to 5.25 gpm per sq ft. Symbol ○ indicates sand particles of 0.936 mm; symbol + indicates sand particles of 0.532 mm. Data from Stanley (5).

that floc penetration is governed by the relation:

$$K = \frac{Vd^3h}{L}$$

in which K is an index of filterability. It is proposed that the earlier relationship, which did not include a velocity term, be abandoned and that $\frac{Vd^3h}{L}$ be designated the filterability index.

The similarity of the relationship between frictional resistance in porous beds and the velocity through the bed to the relationship between floc penetration and pore velocity suggests that turbulence may play a part in the behavior of granular beds in filtration of strongly flocculated water. This may account for the discrepancies between results reported by various investigators.

For the most common situation of strong flocculation, floc penetration appears to be related to the expression:

$$K_1 = \frac{Vd^2\sqrt{h}}{L}$$

It will be observed that, in either case, the penetration of suspended matter into the bed seems to be directly proportional to the filtration rate. It would therefore appear that higher filtration rates could be used with proportionately thicker beds, finer sand, or lower head losses, without impairing filtered-water quality.

To determine the limiting values of the filter design parameters requires routine operation of one or more filters on a given water to beyond the breakthrough. This may be done through either high-rate operation or through the use of coarser sand, shallower beds, or higher terminal head losses. It can safely be attempted only at public water supply plants exercising accurate and frequent measurement of filtered-water quality from individual filters and completely trustworthy disinfection of the water, or at plants that are not producing drinking water. Such trials should continue through a number of

years in order to gain data on the intensity, frequency, and duration of weak flocculation. Particular attention will need to be paid to cold-weather conditions, when the weakest floc may be encountered. The improvement of filter design practice awaits the gathering of these data.

Filterability Index Values

The most critical value for the filterability index encountered was 0.4, when V was expressed in gallons per minute per square foot, d in millimeters, h in feet of water, and L in feet. Additional tentative values may be derived from information given above. These values, as shown in Table 2, are for filtered water free of flocculated turbidity.

TABLE 2

Values of Filterability Index K for Various Waters and Conditions

Type	Conditions	Value of K
I	Raw water difficult to coagulate, average pretreatment facilities and operation	0.4
II	Raw water not hard to coagulate, average pretreatment conditions	1.0
III	Average raw water, high-grade pretreatment facilities	2.0
IV	Average raw water, high-grade pretreatment facilities plus close technical supervision and activated-silica treatment	6.0

The values of the filterability index given in Table 2 do not take into account variations in bacterial loading. They are based on maintaining the ability of the filter so that it can uniformly produce high-quality water. As suggested above, it would appear valuable to assemble additional data on seasonal and annual variance of the filterability index at other plants.

The values given above emphasize the desirability of caution in increasing filtration rates. Where low filterability indexes occur, the situation is not impossible, however. Low values can be met by backwashing filters at low head losses. Fortunately, too, low filterability indexes tend to occur at those seasons when filtration rates are low because of decreased demand.

Conclusions

The trend in filtration practice has been toward increased rates of filtration, increased sand sizes, and thinner beds. Some municipal plants, although designed for 2 gpm per square foot, are successfully operated at rates as high as 3 gpm per square foot during periods of high demand, and especially when the water is warm. The studies analyzed in this report bring out relationships between sand sizes, filtration rates, terminal losses of head, and bed thickness. Any one of these design parameters may be altered by suitable adjustment of one or more of the others.

The indiscriminate adoption of filtration rates above 2 gpm per square foot should not be encouraged. Rates above 2 gpm per square foot may be adopted, however, if there has been sufficient experience to establish limiting values

of the filterability index (preferably 5 years or more) obtained by operation in the plant under consideration or in nearby plants filtering water pretreated in a manner similar to that proposed, using water from the same source.

The criterion upon which filter design may be based to meet the most critical water quality requirements is the filterability index, which has been modified to be defined as:

$$K = \frac{Vd^3h}{L}$$

Limiting values of this index may be determined by operation of one or more filters over a period of a number of years to the point at which actual breakthrough of turbidity may possibly occur.

Acknowledgments

This study was brought to its present state under the auspices of the Subcommittee on Water Supply, Committee on Sanitary Engineering and Environment, Division of Medical Sciences, National Research Council. E. W. Moore is chairman of the subcommittee; he and the other subcommittee members have been unflagging in their encouragement, aid, and stimulation of the preparation of the study.

The writer is also indebted to the following for their critical reviews of the material while it was in preparation: Norman E. Jackson, John C. Geyer, Harry N. Lowe, John R. Baylis, Richard Hazen, and Gordon M. Fair. It is not intended to imply that the study states their views. They have all contributed ideas, however, that helped the author form the views presented.

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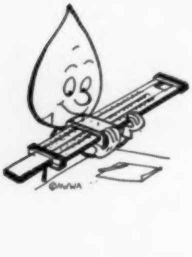

Protection of Utilities Against Permafrost in Northern Canada

Stanley S. Copp, Carl B. Crawford, and John W. Grainge

A paper presented on Apr. 23, 1956, at the Canadian Section Meeting, London, Ont., by Stanley S. Copp, Sr. Engr., Dept. of National Health & Welfare, Vancouver, B.C., Carl B. Crawford, Asst. Research Officer, Div. of Building Research, National Research Council, Ottawa, Ont.; and John W. Grainge, Sr. Engr., Dept. of National Health & Welfare, Edmonton, Alta.

DEVELOPMENT of the Canadian northland has progressed so rapidly during the last decade that the problem of providing the facilities of modern living in the North has become one of the greatest current engineering problems. Owing to unusual features of the climate and the ground, it is apparent that engineering practices of more temperature regions cannot be applied without modification. This is particularly true of the design of water and sewer services in regions of "permafrost," or perennially frozen ground (*1*). Descriptions of such systems will be found on p. 1166.

When it became necessary in 1945 to relocate the town of Yellowknife, Northwest Territories, Can., because of a rapidly increasing population, a recirculating water distribution system was designed similar to the one used successfully at Flin Flon, Man., since 1931. To the authors' knowledge, the Yellowknife installation is the second of its type to be employed in Canada. In this system water is preheated, circulated through dual mains and dual house service connections, and a portion is returned to the pumphouse (*2*).

Yellowknife

Yellowknife is located 185 miles south of the Arctic Circle and about 600 miles north of Edmonton, Alta. It has a population of about 2,500. The construction of complete municipal services at Yellowknife provided an ideal opportunity to study ground temperatures around water and sewer pipes at a typical northern town. In 1951, therefore, the Public Health Engineering Division of the Department of Health and Welfare, in cooperation with the Division of Building Research of the National Research Council, began such a study. Primarily, the field work was planned to obtain ground temperature data to assist in the design and operation of the Yellowknife system and other similar water and sewer systems. It was expected that these measurements would reveal the effect of the distribution system on the natural temperature and thermal properties of the ground, and provide an evaluation of the insulating effect of ground moss when placed around the pipes. Also, because of the lack of this formation, calculations such as the required boiler horsepower of the heating plant could not be estimated with the normal degree of accuracy. The study was coordinated with the general study of ground temperatures being carried out by the Division of Building Research. As part of that study, various stations for ground tem-

perature measurements are located throughout Canada.

Factors Affecting Temperature

It is convenient to consider that the temperature beneath the surface of the ground is dependent on the air temperature; it has been shown, however, that this is not merely a simple relationship based on the heat flow theory (*3*).

It is difficult to assess the effect on ground temperatures of such other meteorological elements as wind, relative humidity, and precipitation. It is known that the nature of surface cover will have a major effect on subsurface temperatures. Snow cover, for instance, may reduce frost penetration by several feet (*4*). Color and vegetation will significantly affect the radiation exchange at the surface. The nature of the soil itself will influence its temperature, primarily because of the usual variations in water content between soils. Fine-grained soils generally have a high water content while coarse soils contain little water. (It is not unusual for clays to be made up of $\frac{2}{3}$ water by volume, and nearly all natural clays will contain more than $\frac{1}{3}$ water by volume.) Considerably more heat must therefore be extracted from a wet soil than from a dry soil in lowering the temperature; because of the latent heat of fusion which is liberated, much additional heat extraction is necessary before water in soil will freeze. For these reasons, too, the frost line will move more rapidly and to greater depths in dry granular soils than in wet soils. Because the water in soils has such a great effect on the thermal properties, it is evident that the water content will govern ground temperatures to a large degree. It is very difficult, however, to take water content into account, a further complication being introduced by its seasonal variation.

Many attempts have been made to compute the depth of frost penetration (*5–7*) but the reliability of these methods has been restricted largely because of the difficulty of evaluating theoretically the thermal properties of the soil and because these properties vary with changes in water content. This has resulted in empirical studies that have established an approximate relationship between frost penetration and air temperature (*8*). An approximation of the average frost penetration anywhere in Canada can be obtained by selecting the appropriate "freezing index" and referring to the established relationship (*9*). The "freezing index" is the cumulative total of degree-days * below freezing during any winter.

The mean annual air temperature for Yellowknife, which is situated at latitude 62°30′, is 22°F. Permafrost may be encountered at 1–10 ft below the surface, depending on vegetation, and is reported to extend to a depth of more than 150 ft (*10*). The mean annual precipitation is 10 in. with sparse snowfall. The freezing index over a 10-year period has varied from 5,370 to 7,450 degree-days with an average of 6,590. This is about the same as for Churchill, Man.; it is about double the values for Winnipeg and Regina, and nearly three times the value for Ottawa. According to the empirical relationship based on freezing index, the seasonal frost penetration in Yellowknife should be on the order of 9 ft. The accuracy of this relationship for permafrost regions may be questionable.

* A degree-day represents 1° of declination from 32°F in the mean daily air temperature.

Water and Sewage System

Domestic water for Yellowknife is drawn from Yellowknife Bay, a large arm of Great Slave Lake. Lake water is discharged by float-controlled primary pumps into a 6,000 gal makeup tank in the pumphouse. Water in this equalizing tank is chlorinated and is then mixed with the reheated return water from the distribution system. This mixture is discharged to the distribution system through high-lift pumps. All water lines are of cast iron. The main from the pumphouse is 8 in. in diameter and the return is 6 in. in diameter, while laterals are 6 in. with 4-in. diameter returns. House connections are ½-in. copper pipe, the lead to the house being taken off the main and the return from the house connected to the return main. An orifice disc containing one or more holes is located in a fitting within the house on the return house connection. The water distribution system is graded so that it can be drained in the event of a prolonged breakdown. It has a minimum cover of 5 ft 6 in. All water and sewer lines are insulated with moss for about a foot above and on the sides of the pipes, and about an inch below the bottom of the pipes.

The sewer system is laid with a minimum earth cover of 7 ft 6 in. and a minimum grade of 0.40 per cent. The sewers are 6 ft horizontally from the water mains at a grade which is 2 ft lower. All sewer pipe is asphalt-dipped corrugated steel, with an extra float to smooth the invert. Eight-inch sewer laterals are intercepted by a 10-in. line to the sewage disposal plant.

The summers of 1947 and 1948, when construction was carried out, were cold and wet, and this resulted in a slow thawing of the permafrost. Because of the frozen soil below, ground water moving through the fine sand of the thawed layers was prevented from escaping, and it accumulated in the trenches, causing construction difficulties of such magnitude that it was necessary to lay a system of underdrains beneath more than half the sewer lines. This consisted of 13,000 ft of 6-in. corrugated steel pipe perforated along the invert. With the removal of the ground water, trenching proceeded normally by dragline, power shovel, or backhoe. Trenching machines could not be used in the fine sandy soil.

Temperature Measurements

It was decided to obtain soil temperature readings on a main water line and a lateral, and on a sewer interceptor and lateral. The effect of the moss insulation around pipes was to be determined, so moss was stripped from 12 ft of water pipe and, at the center of the stripped section, thermocouples were placed about the pipe. Fifteen feet further along on the same pipe, where the moss was left intact, another set of thermocouples was placed in the same relative position to the pipe. A three-pen recording thermometer was located between the water and sewer laterals to provide additional temperatures and to check on the thermocouple readings.

Ground temperatures were measured at seven stations. The exact locations of the thermocouples at Stations 1–3 are shown in Fig. 1. Thermocouples are in the same positions relative to the pipe at Stations 4–6.

The temperature-sensing bulbs of the three-pen recording thermometer at Station 1 are located at depths of 4, 7, and 10 ft below lane level, midway between stations 2 and 3, as

shown in Fig. 1. The graveled lane is well traveled during both summer and winter, and the snow is plowed as required, although the compacted snow reaches a depth of 4 in. or more.

At Station 2, thermocouples are located above and below a moss-covered sewer lateral serving one block of government-owned houses. The lateral is 3 ft to the side of the centerline of the lane and 3 ft 4 in. from a vertical line through the recording thermometer bulbs.

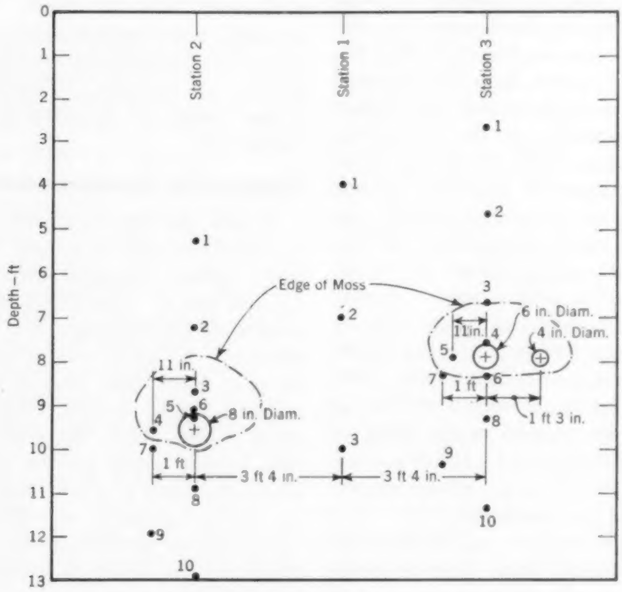

Fig. 1. Ground Temperature Points at Yellowknife

Stations 1–3 are shown. Station 1 was between the water and sewer laterals, Station 2 at the sewer lateral, and Station 3 at the water lateral.

At Station 3, thermocouples are located around a moss-covered water lateral and therefore 6 ft 8 in. horizontally from the sewer lateral at Station 2 and 3 ft 4 in. from a vertical line through the recording thermometer bulbs.

At Station 4 the moss is stripped from 12 ft of the water supply line to the town. This line is of 8-in. cast-iron pipe with a 6-in. return laid 15 in., center to center, from it. The main runs below an untravelled road 3 ft to the side of the centerline. Drifted snow may reach a depth of 12 in. or more over the roadway.

Station 5 is on the same water main 15 ft along the pipe, but at a point where the moss insulation around the pipe is undisturbed. Roadway conditions are the same and thermocouples are located in the same relative positions as at Station 4.

At Station 6, thermocouples are placed around the 10-in. moss-covered sewer main leading to the sewage treatment plant. The main is 3 ft from the centerline of a continuously travelled roadway. The snow may be compacted to a depth of 6 in. or more.

Station 7 is in moss-covered undisturbed ground in a vacant lot and is used as a control. Thermocouples are located at depths of 2.3, 4.3, 6.3, and 8.3 ft. Undisturbed snow depth may reach 12 in. or more.

Instrumentation

Temperatures were measured at Station 1 by a three-pen mercury-bulb recording thermometer with temperature-compensated leads and operated by an 8-day spring-driven clock mechanism. At all other stations temperatures were measured periodically with a portable potentiometer connected to copper-constantan thermocouples. As first installed, the copper wire of each copper-constantan duplex thermocouple wire was severed above ground and joined to the wafers of a ten-point switch. The reference junctions of each circuit were bound together and placed in a crushed-ice and water mixture during reading and the potentiometer was connected to the switch poles through telephone jacks. Each switch was mounted in an insulated box and protected by a small unheated shelter which was mounted several feet above ground near the roadway.

Owing to difficulties in obtaining accurate readings during winter under severe weather conditions and because of the impossibility of operating the potentiometer satisfactorily inside a heated vehicle, it was considered necessary to improve the observation facilities. During the fall of 1954, various modifications in instrumentation were carried out, and improved switches were installed. The altered circuit arrangement is shown in Fig. 2. This arrangement has the advantage of reduced resistance, permitting increased sensitivity, and a less cumbersome reference junction. Switch boxes for stations 2 and 3 were installed with the recording thermometer in a heated garage and a special heated building was constructed for the switches for stations 4–7. The arrangements provided protection for the instruments

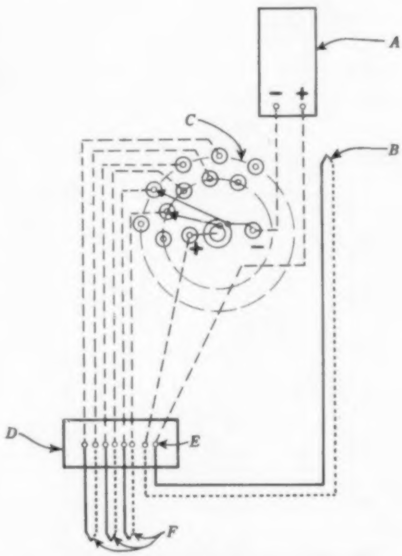

Fig. 2. Revised Thermocouple

Letters indicate the following: A—*potentiometer;* B—*reference junction;* C—*two-pole rotary switch;* D—*junction box (of cast iron, sealed and buried to at least 3-ft depth);* E—*compensating junction; and* F—*measuring thermocouples. The solid line indicates constantan duplex wire, the dotted line copper duplex wire, and the dashed line copper lead wire.*

and permitted ideal conditions for operation of the potentiometer at all stations, which resulted in improved accuracy in the readings.

The selection of instruments for ground temperature measurements is a special problem. The instrument must be rugged, accurate over a long period, reasonably cheap, and easy to operate. The mercury bulb type of instrument is often used because of its simple operation. It can be reasonably accurate but it may get out of adjustment during prolonged field use. Also, it is too expensive for extensive use. Various resistance-type instruments are available, but they too are expensive and their stability is questionable over long periods. Thermocouples are very cheap, rugged, and reasonably accurate, but the potentionmeter is a sensitive instrument which requires care in operation and reasonably stable temperature conditions. Thermocouples can be calibrated to yield highly accurate measurements but, in normal field use, errors caused by contact potentials, stray ground potentials, reference temperature errors, and positioning errors, as well as personal errors, may combine to give a substantial absolute error in measurement. It is thought that the measurements reported in this paper will be within a maximum error of ± 1°F; where gross errors were evident, results were discarded. In general, monthly average temperatures were used, and these showed trends which seem quite accurate.

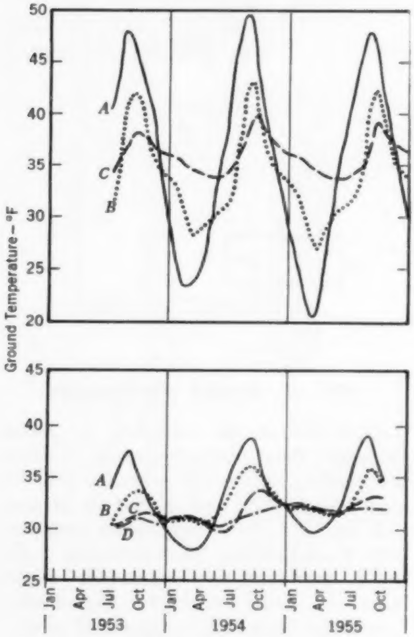

Fig. 3. Ground Temperatures, 1953–55

The top portion is for Station 1. Line A *is for temperatures at a depth of 4 ft; Line* B, *at a depth of 7 ft; and Line* C, *at a depth of 10 ft. The bottom portion is for Station 7. Line* A *is for a depth of 2.3 ft; Line* B *for a depth of 4.3 ft; Line* C *for a depth of 6.3 ft; and Line* D *for a depth of 8.3 ft.*

TABLE 1

Grain Size of Soils at Yellowknife

Station	Depth Represented	Grain-Size Distribution—%			
		Gravel	Sand	Silt	Clay
1–3	Fill above pipes	7	30	50	13
1–3	Below pipes	0	50	50	0
4, 5	Fill above pipes	10	70	20	0
4, 5	Below pipes	0	33	42	25
6	Fill above pipes	11	48	32	9
6	Below pipes	0	46	47	7
7	1.0–2.5 ft	0	77	23	0
7	2.5–8.0 ft	0	23	51	26

Temperature Observations

The new townsite of Yellowknife was originally moss covered. The sub-

soils consist generally of fine sandy silts and silty sands. The average grain size of soils from certain locations are shown in Table 1. In the developed areas, the moss cover has been removed and the road bases consist of a 6-in. layer of sandy gravel. The effect of removing the moss cover is clearly shown in Fig. 3. At Station 1, under a well traveled lane, the annual temperature variation at a depth of 4 ft is nearly 30°, while under the undisturbed moss at Station 7 the variation at 4.3 ft is less than 6°. Similarly, at other depths, the seasonal variation under moss cover is much less than in the disturbed ground and the temperature gradient during winter is very slight, indicating relatively little heat loss from the moss-covered ground under natural snow cover.

The effect of removing the moss cover and the installation of services is further illustrated in Fig. 4. At Station 7 the natural permafrost condition is maintained with the active layer reaching a depth of 6 ft in 1953 and 9 ft during the unusually warm summer of 1954. At Station 1 the permafrost has been destroyed to a considerable depth, the frost now penetrates seasonally, as it does in more temperate regions. As will be shown later, the frost penetration at Station 1 was reduced by the proximity of the water and sewer services.

Annual temperature variations at Station 2 are shown in Fig. 5. Lines *C* and *B*, which are inside and just above the pipe, follow one another closely. This fact, together with the observed warming from December to February (at a time when cooling should be occurring at the maximum rate), indicates the effect of the sewer on ground temperatures. The warming apparently is quite effective 2 ft above the pipe at *A*, as shown in Fig. 5.

Figure 5 also shows some interesting temperature comparisons between Station 1 (located between water and sewer lines) and Station 3 (water lateral). At Station 1, at the 7-ft depth, annual variation is about 27°–43°F and at Station 3, at the 7.6-ft depth (top of pipe), it is about 36°–50°F. The average annual temperature near

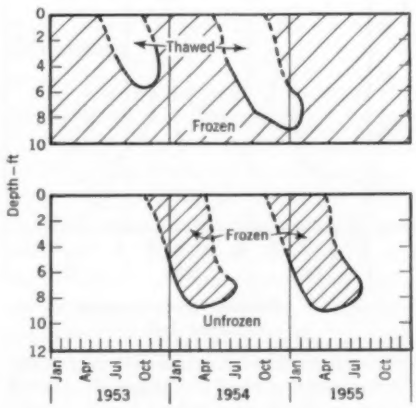

Fig. 4. Freeze and Thaw Penetration

The top portion shows the thaw penetration at Station 7, in undisturbed moss-covered ground. The bottom portion frost penetration at Station 1, under a snow-cleared lane.

the pipe is about 7° warmer than at Station 1. A leveling off and an actual rise in the temperature of the pipe during winter are shown. The difference between these two curves can be attributed mainly to heat loss from the water supply. The temperature profiles shown in Fig. 6, and the isotherms of Fig. 7, are further evidence of the great effect on ground temperatures of the water and sewer systems.

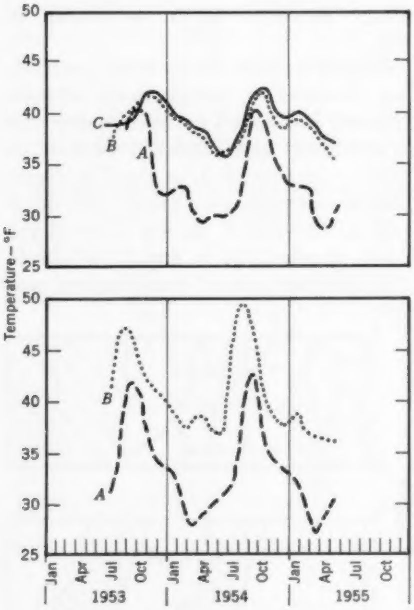

Fig. 5. Ground Temperatures at Stations 1–3

The top portion shows data for a sewer lateral under a road at Station 2. Line A *is for a depth of 7.3 ft, Line* B *for 9.1 ft, and Line* C, *for 9.3 ft (inside the pipe). The bottom portion of the figure indicates: Line* A, *Station 3 at a depth of 7.6 ft; and Line* B, *Station 1 at a depth of 7.0 ft.*

For convenience of comparison, monthly average ground temperature profiles for stations 2–7 are shown in Fig. 5. Points on the curves show the location of thermocouples. At Station 2, one thermocouple was led through a hole drilled in the top of the sewer pipe and it, therefore, measures the air temperature above the sewage. This temperature, on a monthly average basis, ranges from 35°F to 42°F, while the temperature of the sewage reaching the treatment plant in winter is over 40°F and may jump to 55°F for a short time in the evening. At Station 3, the sharp temperature rise in the pipe during summer is attributed to a rise in the lake water temperature to about 55°F. Station 6 is an installation on the sewer interceptor leading to the sewage plant. Station 7 is an installation in undisturbed ground.

Some interesting temperature comparisons are available in Fig. 5. Temperature variation at Station 2, for instance, is much less than at Stations 3 and 6. This is probably because of the smaller flow in the sewer lateral and the greater soil cover over this pipe. The great difference in temperature variation between the natural ground (Station 7) and service installations is particularly striking. The increased temperature amplitude is caused mainly by removal of the moss cover and probably by a change in soil moisture conditions, but the distortion of the temperature gradient curves obviously results from heat exchange with the water and sewer pipes.

The moss insulation was stripped from a 12-ft length of water main at Station 4 and was left untouched at Station 5. The temperature profiles show that they are practically identical through the year, suggesting that the moss insulation is of little or no value. Other insulating materials, such as fibre-glass, insulated pipe, and wood-stave pipe, might have given other results under similar operating conditions. It should be noted that the value of a material as an insulator depends more or less directly on the volume of entrapped air in the material. If the material becomes wet and the voids filled with water, the insulating properties are lost.

Upon comparing the temperatures of stations 4 or 5 with those of Station 3, the effect of volume of flow in the pipe on ground temperatures is again seen. The frost penetrated to within 3–4 in. of the water lateral, but only to about 12–13 in. above the water main, even though the main had less soil cover than the lateral.

Ground temperature isotherms on Mar. 6, 1955, for stations 1–3 are shown in Fig. 7. These illustrate the effect of the water and sewer pipes in reducing frost penetration at Station 1.

Annual ground temperatures are shown in Fig. 8. Curve *A* shows that the 1954 annual temperature at Station 7, where the ground is undisturbed, is almost constant to a depth of 8 ft. Curve *B* shows the 1954 annual temperature at Station 4, which is typical of all other stations. Curve *C* illustrates the annual temperature under a snow-cleared surface at Ottawa and curve *D* is the annual temperature under natural snow cover. With respect to surface conditions, curves *A* and *D* are equivalent, and curve *B* corresponds to curve *C*. Other investigations have also revealed that annual ground temperatures are practically constant to shallow depths and this temperature is always greater than the annual air temperature. (At Ottawa it is more than 6° greater and at Yellowknife it is about 11° greater.) This difference has been attributed to the complex heat exchange between the ground surface and the atmosphere, and to the effect of the snow cover (*3*).

The difference between curves *A* and *B* of Fig. 8 shows there is heat loss to the ground from the water main. Actually, the boundary conditions resulting in Curve *A* are different from those shown in Curve *B* in that, at Station 7, undisturbed snow cover is present and moss cover has not been removed. For direct comparison, a reference station under a cleared roadway without pipes would be necessary. Based on differences between curves *C* and *D*, it can be assumed that the true annual temperature difference caused by heat exchange with the pipes (curves *A* and *B*) would be somewhat greater than that shown.

Discussion

No attempt has been made in this paper to relate, through heat conduction theory, the measured ground temperatures at Yellowknife to other variables. Any rigorous approach based on heat transmission is extremely difficult and very involved in view of the impossibility of obtaining simple mathematical expressions which adequately describe the variations in the climatic elements with time. Further formidable complications are introduced by snow cover, which may vary in depth and in thermal properties, by the soil, including moss cover, with its accompanying water, whose thermal properties may vary with time and location. In any field experiment designed to provide for the measurement of all the pertinent factors, it would be necessary to evaluate the thermal properties of the soil, to obtain accurate flow, temperature, and heat input data for the water and sewer lines, and to record continuously the pertinent climatic elements.

The above complications are inherent in any exact approach to the design of systems. There are grounds for optimism, however, and preliminary studies at Yellowknife as well as at

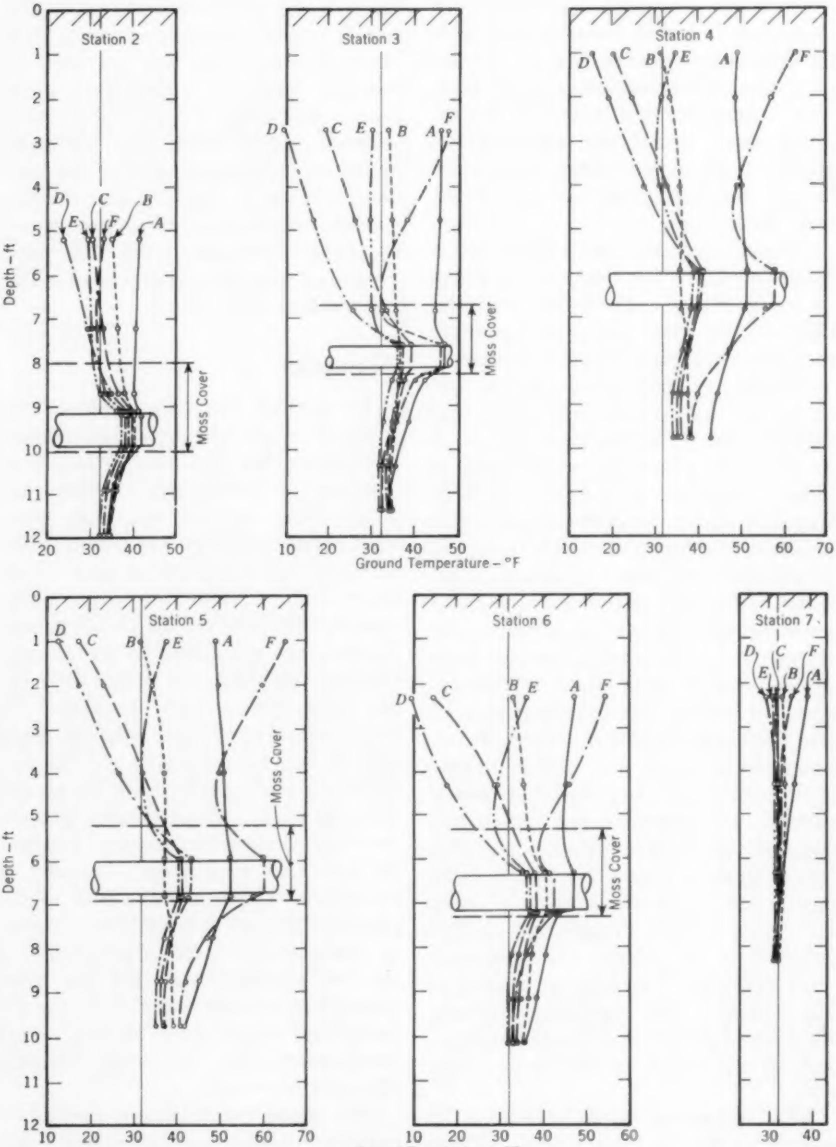

Fig. 6. Monthly Average Ground Temperatures at Yellowknife

Lines show data for the following: A—*September 1954;* B—*November 1954;* C—*January 1955;* D—*March 1955;* E—*May 1955; and* F—*July 1955.*

other places show that, by introducing approximations in the theoretical approach and by adjusting the necessary coefficients on the basis of field measurements, manageable and useful relationships can be established. Much work will be required before this can be achieved.

in the design of other systems. Specifically, the data presented illustrate the effect of a heated water supply on ground temperatures and frost penetration and add to the general information on ground temperatures. They show the great effect of removing moss cover in permafrost regions and yet indicate

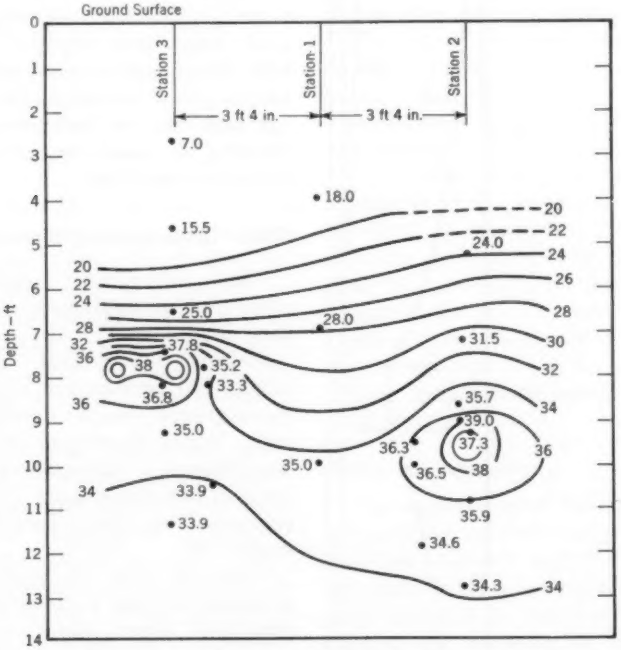

Fig. 7. Annual Ground Temperature Variations at Yellowknife

Approximate ground temperature isotherms are given for Mar. 6, 1955, at Stations 1–3. Temperatures shown are in degrees Fahrenheit.

In the meantime, there is only very limited experience for use as a guide in laying out systems in permafrost. Ground temperature data such as those which have been presented are useful in evaluating protective measures taken against freezing in a particular system, and they provide some guidance

that moss has little insulating effect when placed around pipes. This evidence supports the contention that wet moss is not a good insulator in the normal thermal sense; when moss is present at the surface, however, it protects the permafrost, through evaporation, by redirecting the heat which it re-

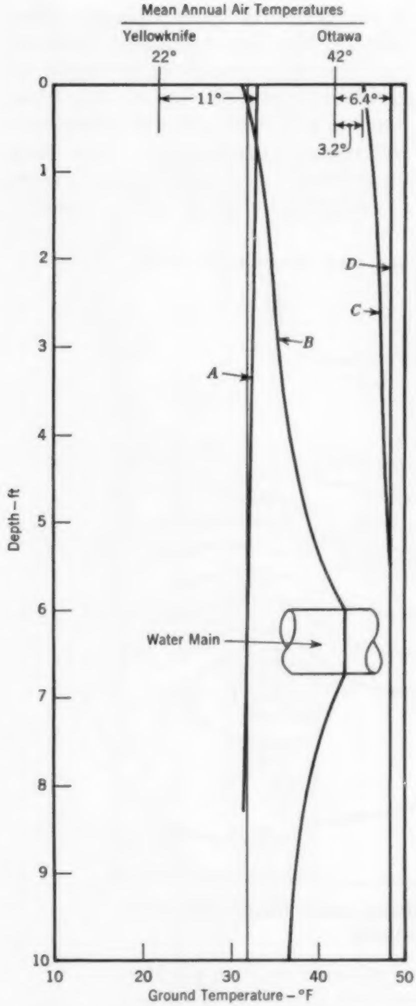

Fig. 8. Relationship Between Air Temperature and Annual Ground Temperature

Lines indicate data for the following: A—*undisturbed ground at Station 7, Yellowknife;* B—*Station 4, Yellowknife;* C—*pit under a snow-cleared surface, Ottawa; and* D—*under natural snow cover, Ottawa. Temperatures are annual temperatures for 1954.*

ceives during the summer from above. Probably the most important feature of these observations is that they give a general picture of the changing thermal regime in the ground when municipal services are installed in a region of permafrost. The two main problems—the amount of heat which may be required for a particular installation to keep pipes from freezing, and the most economical depth at which to bury the pipes—are not yet specifically solved. This investigation, however, has provided the background for the planning of future research on these important questions.

Other Underground Systems

There follow brief descriptions of other underground systems in permafrost conditions.

Aklavik, Northwest Territories. About 200 persons are served by an above-ground summer distribution system. Water is pumped from a small lake through a diatomaceous earth filter, and then chlorinated. The system operates from about June 1 to September 20.

Churchill, Man. A large army base at Churchill has a steam-heated, recirculated, water system consisting of a wood-stave supply main and distribution system consisting partly of utilidors, which are large insulated ducts carrying water, sewer, and steam lines, usually above or just below ground level.

Dawson City, Yukon Territory. This town, with a population of 300, has lines of wood-stave pipe in gravel, at a depth of 1.5 ft. The pipe was installed in about 1900, the time of the Yukon gold rush. Water is pumped from the Klondike River or wells, heated to 42°F, and bled to sewers

through partially opened $\frac{3}{8}$-in. valves in residences and at the ends of mains. The temperature of the bleeder water at the end of the system reaches 36°F in winter. The temperature drop of 6° is not surprisingly great when it is considered that nearly 2 mgd are pumped through the small system. The system is owned and operated by the Dawson City Water and Power Ltd.

Fairbanks, Alaska. Fairbanks, with a population of 7,500, has a single-main wood-stave pipe system with dual copper service connections. It was installed in 1954. The 3-fps velocity in the mains provides sufficient velocity in the dual house connections to prevent freezing, even though they are only at a depth of 6 ft. Cost of this system was $2,600,000, while a dual-main recirculating system, such as at Yellowknife, was estimated at $3,-500,000 and a steam tracer system at $4,700,000.

Flin Flon, Man. Flin Flon, with a population of 10,000, supplies water which is preheated to 40°F and recirculated as at Yellowknife. The return water is maintained at 38°F. Adequate circulation is considered to be the critical factor. Water and sewer lines are laid together, often at shallow depths or even above ground, in wooden boxes insulated with shavings. The supply pressures at the two pumping stations are 70 and 100 psi, respectively, and the return pressures are 8–20 psi less than that, depending on the freezing hazard. To heat the water, an average of 1 gal of bunker oil is burned for every 1,000 gal of water pumped during the heating season. This represents approximately the amount of heat required to raise the temperature of 1,000 gal of water by 8°.

Fort Smith, Northwest Territories. Fort Smith has a population of 400. A single-main system, laid at a depth of 8–10 ft, distributes heated, treated, river water. To maintain movement, water is bled at dead ends. A three-bulb recording thermometer was installed at a Y branch in the system and has been helpful in determining which pipes are in danger of freezing.

Whitehorse, Yukon Territory. A limited portion of Whitehorse (population 2,000) is served by the army water supply system, which consists of pumping unheated water from McIntyre Creek through a main feeder at a depth of 6–7 ft, and then through bleeders. A new system is now being installed and water will be pumped from the Lewes River or wells through a distribution system at a minimum depth of 9 ft. House connections will have orifice bleeders $\frac{5}{64}$ in. in diameter, through which water will discharge to the sewer.

Acknowledgments

The authors wish to extend their appreciation to their many colleagues who have assisted with the field work and with the analysis represented in this paper and to J. R. Menzies for his support in the initiation of the project. Special appreciation is due R. F. Legget and N. B. Hutcheon for their active interest and specific suggestions. This paper is presented with the approval of J. R. Menzies, Chief of the Public Health Eng. Div. of the Federal Dept. of National Health and Welfare, and R. F. Legget, Director, Div of Building Research, National Research Council.

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Correction

The paper "Pump System Analysis and Planning Methods Used in the Los Angeles City Water System" by Roland Triay Jr. (June 1956 Journal, Vol. 48, pp. 629–653) contained several editorial errors. On p. 640, the check valve symbols in Fig. 5 should point North, as they are fed from another system on the south side. In Fig. 8, on p. 646, the single-hatched area *below* the 1,700-ft hydraulic grade line (operating range) should be crosshatched. Similarly, in Fig. 9, on p. 648, the crosshatched area *above* the 1,700-ft line should be single hatched. Also, in Fig. 10, on p. 649, all legends reading "Total Output" should read "Consumption."

Use of Activated Silica at Quincy, Ill.

Orville J. Smith

A paper presented on Mar. 22, 1956, at the Illinois Section Meeting, Chicago, Ill., by Orville J. Smith, Chemist, Quincy Water Works Com., Quincy, Ill.

A GREAT deal of informative material has been prepared on the use of activated silica in water plant operation since its first successful large-scale use by John R. Baylis in 1937. Baylis activated a soluble silicate by diluting it to 1.5 per cent SiO_2 and neutralized 85 per cent of the alkalinity with concentrated H_2SO_4 until an alkalinity of 1,100–1,200 ppm resulted. After aging for 2 hr, the solution was finally diluted to 0.6 per cent SiO_2. The resulting dilution proved to be a valuable aid to coagulation.

The object of this article is to report the various advantages available for plant treatment by activating silica on a continuous basis with chlorine.

Theory

Activation, or colloidal micelle growth, occurs through a release of silica from sodium silicate. The degree of activation is controlled by [1] the starting dilution of the sodium silicate, [2] the amount of time set for aging, [3] the amount of neutralization, and [4] the temperature of the makeup water. Properly activated silica assumes the characteristics of a negative colloid. Certain alum color or clay flocs carry an excessive positive charge. Thus, by striking a proper balance between the charges, it is possible to obtain a neutralization between them and, as a result, depeptize the floc particles. This results in a tougher and denser floc which settles very rapidly.

Background

In the years since the first use of activated silica for water plants, many other chemicals besides H_2SO_4 have been utilized for partially neutralizing alkalinity in preparing sodium silicate for use as a coagulant aid. Some of these chemicals are aluminum sulfate, sodium bicarbonate, sodium aluminate, sulfur dioxide, carbon dioxide, ferrous sulfate, and ammonium sulfate. Any of these chemicals, when used in correct proportions, will provide activated silica.

The first method of producing activated silica was the batch method. This was successful, but it involved a large storage problem because of dilution, neutralization, and aging. Continuous-activation methods which reduce this storage problem are now available.

Conditions at Quincy

The general treatment of water in Quincy, Ill., is prechlorination, and then coagulation with alum. After 10 min of over-and-under mixing, the water is allowed to settle in a plain sedimentation basin for a period of 45 min. It is then softened, clarified, recarbonated, and allowed to settle for 2 hr before filtration.

At Quincy, the most difficult coagulation problem occurs in the early spring of the year, resulting from the thawing and melting of snow on the upper Mississippi watershed. By the time the water reaches Quincy, the heavier particles of sediment have settled out and the continuous turnover on passing through twenty dams on the upper stretches of the river has caused any remaining sediment to be very fine, colloidal, and high in color. The turbidity is 300–500 ppm with a low alkalinity of 80–110 ppm and a low hardness of 110–130 ppm.

Because the magnesium content of the water is low at this time, no appreciable help can be expected from the softening reactions. To some extent, this condition is repeated in June, when the Mississippi generally has a flooding condition because of the spring rains in Iowa. The turbidity at this time is usually higher, but it is much easier to handle because of the larger particle size. Presedimentation will reduce the turbidity by almost 50 per cent at this time, while little effect is shown by presedimentation in the early-spring condition. This problem was formerly met by adding soda ash in conjunction with alum and chlorine, thus producing an artificial alkalinity with which the alum could react. This method proved fairly effective, but high coagulant dosages of 50–80 ppm of alum and 10–15 ppm of soda ash were sometimes necessary.

Activated silica had been used occasionally in jar tests which indicated that it might be an effective coagulant aid in working with the difficult conditions.

Use of Activated Silica

Activated silica was introduced at the Quincy plant in October 1953. Equipment for chlorine–activated silica treatment was installed on a trial basis at this time. Because plant treatment already included prechlorination, no additional chemical costs were incurred.

In the process of activation, 41°F sodium silicate is pumped at a preset rate into a baffled mixing chamber, where it is activated by a controlled volume of chlorine solution. It is then aged a short time, diluted, and ejected to the point of application. It was found most satisfactory to make the silicate application in conjunction with prechlorination in the first over-and-under baffle preceding the alum feed.

The results of this treatment can best be judged from the records for March–July of 1952 and 1953, when silica was not used, and for the same months of 1954 and 1955, when it was in use. The average daily turbidities during these two periods were practically identical, reflecting the Mississippi runoff in early spring and the rise in early June. In 1952 and 1953 the daily turbidity averaged 229.5 ppm, and 36.18 ppm of alum and 7.57 ppm of soda ash were used in treatment. The comparative months of 1954 and 1955, when activated silica was used, show an average daily turbidity of 225.5 ppm, and 25.89 ppm of alum and 3.70 ppm of 41°F sodium silicate were used in the plant treatment. This average dosage of sodium silicate represents 1.06 ppm of activated silica. The comparative costs show an overall reduction in chemical treatment of 33.8 per cent for the activated-silica treatment, 20 per cent of which was a saving on alum as a coagulant.

The use of activated silica has been continued on a year-round basis in the plant treatment, an average of 1 ppm activated silica to 10 ppm alum being used. Although the river turbidities are not usually high during the fall and

winter months, this treatment indicates that it is of benefit in coagulating cold waters and the residual silicate carried over is an aid in the reduction and coagulation of magnesium hydroxide in the softening process. A saving of 30 per cent for coagulants, exclusive of soda ash, is indicated by the year-round use of activated-silica treatment.

A condition of the Mississippi River water that has occurred during the late autumns of the past few years is the high pH of raw water. A pH of 9.27 was recorded in the fall of 1954. This was not particularly a problem, because the turbidity was low and coagulation was followed by softening. Only 20 miles downstream, however, in an ordinary coagulation plant, treatment difficulties arose which finally had to be solved by acid treatment before there was proper coagulation. At the time, it was noted in the Quincy plant that the floc formed by the use of 10 ppm of alum and 1.0 ppm of activated silica was small, but this floc completely disappeared when the silica was omitted from the treatment. This was a definite indication that activated silica would extend the flocculation range for alum and, in this case, eliminate the use of an acid treatment.

As is true with most new operations, a few difficulties arose with the equipment used. The baffled mixing chamber caused some trouble because of premature gelation when operated at a high rate, which made it necessary to change the chamber more frequently than the expected one time a day. A second problem of premature gelation was eliminated by the installation of a 5-kwhr heater on the tray water and dilution water supplies. The temperature of these supplies is now held at 60°F during the winter months.

While the general use of silica throughout the year has shown it to be a valuable addition to plant treatment at Quincy, there are certain conditions which arise in water quality that indicate it is not the complete answer to all coagulation problems. One such condition occurred during the last few weeks of February. During the winter, to allow maintenance work on the lock, the pool stage of the water above the Quincy dam had been lowered 4 or 5 ft below the normal stage. It was not brought back to its normal stage until the last week in February. This allowed a lot of trapped water in the sloughs and bays above Quincy to circulate and mix with the river. Because there had been little precipitation during the winter months, the resulting mixture was highly colored. No amount of activated silica up to the machine capacity of 5 ppm seemed to aid coagulation. The only alternative was to cut the silica dosage and resort to a higher dosage of alum.

Conclusion

The author does not wish to imply that, because the activation of sodium silicate with chlorine is the method he used, it is the best or only continuous method for activation. The particular method of activation is best selected by the operator upon review of his own plant treatment.

Regardless of the method, however, the use of activated silica is a valuable addition to plant treatment. At Quincy, it has resulted in a decrease in coagulation costs without sacrificing water quality, and acted as an aid in the softening process for the coagulation and reduction of magnesium, a means of extending the pH range of a coagulant, and a way of securing better overall treatment of cold water.

Discussion

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The behavior of colloids in the concentrations encountered in water treatment is not well enough understood to reduce to a formula the behavior of coagulants and coagulant aids. Even with the instruments available to most water works laboratories, proper coagulation requires judgment based on experience. In many plants, however, the use of activated silica has been found to be a valuable aid in reducing the problems involved in coagulation and clarification.

Related Factors

Activated silica is not a cure-all for coagulation problems. It is a coagulant aid and, as such, is extremely dependent upon the proper application and use of the primary coagulant. An illustration of this is the pH factor. All waters have a definite pH at which optimum coagulation occurs with a given coagulant. This is also the pH at which maximum benefits may be expected from a coagulant aid such as activated silica. It is also true that satisfactory coagulation may be obtained at pH values differing somewhat from the optimum, particularly when a coagulant aid is used, and reduced operating costs may result through savings in alum or other coagulants. This fact is also the basis for the statement that activated silica will extend the pH range of a coagulant.

The period of difficulty which the author described as occurring in February required large alum dosages for coagulation. It is quite probable that the optimum pH had changed considerably. The fact that activated silica did not seem to help is not particularly surprising. There was probably a combination of dosages and points of application that would have been helpful, although practical considerations made this impossible.

Versatility

The wide variety of water treatment processes in which the use of activated silica is beneficial is a tribute to its versatility. Some examples of this will illustrate the variety of treatment processes where activated silica is being used in water clarification.

Iron, present as the soluble ferrous bicarbonate in well water, is removed in one plant by the addition of chlorine-activated silica with enough additional chlorine to oxidize the iron to the insoluble ferric hydroxide. The silica acts as a coagulant aid to the iron floc formed. No other coagulant is necessary.

Another iron removal plant gets its raw-water supply from well fields. Occasionally, some of the iron is oxidized, depending upon which wells are used, and at other times some growths of *Crenothrix* slough from the supply lines. Here again, chlorine is used to oxidize the iron still in solution, 8 ppm of alum is added, and 2 ppm of chlorine-activated silica is used as a coagulant aid. In both of these instances, iron removal is accomplished at a pH of approximately 6.2.

The treatment described at Quincy is a good example of the use of acti-

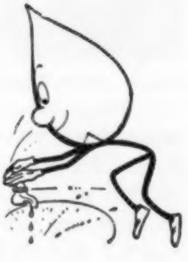
vated silica in the clarification of surface waters. The majority of users fall within this category of treatment. At Quincy, however, the water is later softened. Although dosage rates of activated silica vary from 1 to 13 ppm, the average is 3–4 ppm.

Color is most easily removed by coagulation at a pH of approximately 5. When a coagulant such as alum is added at this pH, a so-called "color floc" is formed, usually small and of insufficient weight to settle rapidly. Activated silica, when used with alum as a coagulant aid, has been found to be beneficial in forming a larger, heavier floc.

When surface water is softened by the lime or lime-soda method, a small amount of coagulant is necessary to coagulate the precipitated carbonates. Activated silica, usually in amounts of 4 ppm or less, has been used successfully in reducing the coagulant dosage needed and, at the same time, in producing a more dense floc. In the event that the raw water to be softened has no turbidity—as is true with well water—and has sufficient magnesium, no coagulation is necessary. Lime, in a sufficient amount to maintain the pH at 10.8–11.0, will cause the precipitation of magnesium hydroxide which acts as a coagulant. Activated silica, acting as an aid to the magnesium floc, is the only other coagulating chemical required.

Conclusion

Within the category of treatment processes listed previously, there are several types which have widely different characteristics and use activated silica for a variety of reasons,—the most common one probably being to produce a quicker settling floc. Multiple benefits, such as those reported at Quincy, are quite common.



Toppling of an Elevated Tank at New London, Minn.

Staff Report

A report based on information supplied principally by Lester Lee, engineer with the firm of Hitchcock & Estabrook, Inc., Minneapolis, Minn.

AT New London, Minn., last February a 50,000-gal elevated steel water tank (Fig. 1), which had been in service only 19 months, toppled for no apparent reason. Having been reported as "leaning slightly" the night before, the spherical tank, 65 ft above grade, tipped over so that its supporting cylinder buckled in two places, leaving the structure in the position of an inverted U (Fig. 2). As there was snow on the ground and 7 or 8 ft of frost in it at the time, only cursory investigations of the failure could be made, but by the end of April a thorough study had been carried out by both the consulting engineers and the firm which had built and erected the tank.*

Neither engineering survey disclosed any defect in the tank itself or in its supporting structure, and, even upended, the tank showed no sign of col-

Fig. 1. New London Tank, Autumn 1954

Fig. 2. New London Tank, February 1956

* Consulting engineers were Hitchcock & Estabrook, Inc., Minneapolis, Minn., and the tank manufacturers and erectors were the Chicago Bridge & Iron Co., Chicago, Ill.

lapse. Footings were 8½ ft in depth in fine white sand that was entirely confined. The structure was designed for 4,000 psf loading, and actual load was less than 3,000 psf plus wind pressure. At the time of the failure there was practically no wind, and even winds of 100 mph would have left total load less than that for which the supports were designed. Still another indication of the soundness of the basic structure was the fact that the concrete footings had not even cracked.

Inquiries by the engineers revealed that a water service connection had been installed to a nearby home in the late fall of 1955. In uncovering this line, they found that at one point it came within 5 ft of the footings, the trench being about 2 ft deeper than the bottom of the footings. As the soil at that point was extremely fine sand—probably passing a 100-mesh sieve—it was assumed that there must have been a flow of sand from under the footings, undermining the structure. This theory was borne out by the fact that the side of the footing nearest the trench had settled approximately 6 in., a condition most unlikely in that area in the absence of undermining. The normal angle of repose of the soil is approximately 30 deg. Trench excavations for sewer and water lines have been unsupported throughout the area, even to depths of 20 ft, and the sides of trenches during construction have stood without caving at slopes of 1½:1 and 2:1. Drainage, too, is excellent, and there was no water on the inside of the footings at any time prior to failure.

The fact that the settlement and toppling of the tank occurred approximately 5 months after the trench had

Fig. 3. Backfill Unconsolidated After 2,000 Years

The soil backfilled into the excavation for the clay pipe drain is still clearly distinguishable from the undisturbed soil. The drain was installed by plumbers of the ancient Babylonian city of Nippur, in approximately 250 B.C. (Photograph courtesy of *CLAY*, published by Clay Sewer Pipe Assn., Akron, Ohio.)

been backfilled and compacted hardly disputes the theory. As a matter of fact, the length of time required to compact soil to its original density is difficult to judge. In one soil in Iraq, 2,000 years seems to be insufficient if the evidence of Fig. 3, showing an excavation next to a clay pipe drain at least that old, is indicative. With no other probable cause to be found, the firm which constructed and erected the tank has been completely absolved of responsibility for the failure. Even so, recognizing the financial significance of the occurrence to a village of 800 population, the firm is erecting a replacement at no cost to the community.

The new tank is to be erected on the same site as the old one, but the hill will be cut down to a point where the bottom of the new footings will be below the present street grade. And provisions are also being made to prevent any excavations within 25 ft of the footings.

Detroit Safety Program

The annual report of the Department of Water Supply, Detroit, Mich., for the period Jul. 1, 1954–Jun. 30, 1955, contains some interesting statistics on safety, as shown in the following table.

Division	Exposure *man-hours*	Number Lost-Time Injuries	Time Charges *days*	Frequency Rate
Maintenance and construction	1,092,324	30	326	27.5
Filtration and pumping	670,692	7	325	10.3
Commercial	367,800	5	86	13.6
Engineering	295,244	0	0	0
Building operations	99,436	0	0	0
Other main office	75,380	0	0	0
Totals	2,600,866	42	737	16.1

As might be expected, the largest source of disabling injuries was the Maintenance and Construction Division. The most serious injuries, however, undoubtedly occurred in the Filtration and Pumping Division, where the average time charge per injury was 46.4 days, as compared to 17.2 and 10.9 days for the Commercial Division and the Maintenance and Construction Division, respectively.

The direct cost per injury averaged $43.09 and covered 240 nondisabling injuries (first aid) and 48 disabling injuries. (Direct costs cover only those items involving the utility, such as compensation and medical costs.) Indirect costs are not given, but are usually considered to be approximately four times greater than direct costs.

Preparation of Engineering Reports for Management

Bruce G. McCauley

A paper presented on Oct. 28, 1955, at the California Section Meeting, Sacramento, Calif., by Bruce G. McCauley, Vice-Pres., Shand & Jurs Co., Berkeley, Calif., formerly Asst. Prof. of Mech. Eng., Univ. of California, Berkeley, Calif.

NO business could function today without written reports, letters, memorandums and other instructions. Writing is the main pipeline through which we transmit our ideas to our superiors, to those who work for us, and to the public in general. It is extremely important, therefore, that written reports effectively communicate the ideas they are intended to convey. This is especially true in a technical field such as the water works industry. All who are associated with this industry must surely be aware of the technical engineering atmosphere which prevails. Unfortunately, many of those with whom business must be transacted daily are unfamiliar with the more technical aspects of the industry.

City councils, county officials, state legislators, even boards of water works directors, all must reach decisions and take action on the basis recommendations which are substantiated in engineering reports. Yet the great majority of these men have no backgrounds in engineering, or even in the sciences—a fact which must always be considered when preparing technical reports.

Not only is it important to prepare reports that are easily understood; it should also be remembered that the costs of preparing reports are continually increasing. The recently announced findings of the Hoover Commission task force studying paperwork indicate that an ordinary 175-word letter costs from \$0.70 to \$2.45 to dictate and prepare. Thus, an engineering report which does not accomplish the purpose for which it is prepared becomes a costly extravagance indeed.

Planning the Report

The first, and perhaps the most important, step in preparing a report is to plan and outline the objective. Having determined the purpose for which the report is being written, the writer must keep that purpose constantly in mind throughout all of the following steps of preparing the report.

At the outset, therefore, it is extremely helpful to state as clearly as possible the major objective of the report. Having correctly determined the subject, the writer should attempt to confine himself to it. Suppose, for example, the purpose is to justify a recommendation for the expansion of water purification facilities. Then, only material pertinent to this objective should be included in the report. If, in the development of the report, the writer encounters other subjects

which appear important, these should be presented in separate reports.

In planning the report, it is wise to consider the reader for whom it is intended. As reports reach the higher levels, those who read them are less interested in the technical details and more interested in their economic and administrative implications. Occasionally, reports are read by civic groups and others who may have no technical background. The effective writer is one who keeps his readers' interests and characteristics foremost in mind. A report that is not understood or does not arouse the reader's interest is generally unfavorably received. The writer, therefore, should always be alert to factors which will maintain the reader's interest. In his investigations the writer should aim at the subject, but in his report he should aim at the reader.

To make the presentation flow smoothly, the writer should consider in advance what he wants to say. A helpful method of developing the framework is to list on a separate 3 × 5-in. card each distinct idea that the writer wishes to present. When all the major points have been covered, including any references to tables, graphs, or illustrations, the cards can be conveniently rearranged into the best sequential order. By such a method the problem of developing the order of presentation into a coordinated pattern is greatly simplified. An outline developed in this manner forms the structural framework on which are hung the various elaborations and amplifications developed later.

Writing the Report

With the outline as his guide, the writer is now prepared to begin the actual writing of the report. There are a number of suggestions which can assist him in preparing an effective, interesting report that will achieve the purpose for which it is intended. Briefly, let us consider some of the more obvious—and more important—guides to effective writing.

In the first place, a report should be written in much the same manner that the author would use for saying it. One of the most frequent faults of technical writing—or, for that matter, of any writing—is the tendency to write in complex, stilted, and unnatural terms. Rather than saying that lining the inside of a pipe with concrete is "conducive to a diminution of corroding transformations," the writer should plainly state that the concrete lining "reduces corrosion." The use of complex words and technical terms may be impressive, but if they confuse and even antagonize the reader, more will be lost than gained. As Robert Gunning, author of *The Technique of Clear Writing* (*1*), has put it: "Write to express, not to impress."

It is generally advisable to keep sentences short. Short sentences serve two useful purposes. First, they are much easier to write than those that become long and involved. Second, they are much easier to understand than those in which the reader becomes lost in a network of qualifying clauses and phrases. Lengthy and complicated sentences rob the reader of valuable time which could be spent studying the report instead of translating it.

Rudolf Flesch, in *The Art of Readable Writing* (*2*), places articles with an average sentence length of 21 words in the "fairly difficult" reading category. This same category included an average of 155 syllables per 100 words and represented the high school graduate reading level.

Even more pertinent to the subject under discussion is the fact that

Flesch found the average scientific article to be in the category which most college graduates would find "very difficult." Furthermore, in terms of human interest such writing was rated in the lowest or "dull" category.

Paragraphs, as well as sentences, should be brief to facilitate rapid comprehension. Only one separate and complete thought should be included in each paragraph. There is also a psychological advantage in short paragraphs: the additional open space which they create provides the reader with a valuable pause in which to collect his thoughts and a feeling of not being overwhelmed by the bulk of words before him.

Specific examples should be used wherever possible. A concrete illustration drives the nail home far more surely and frequently than vague, general statements, which do not have the effectiveness of references to situations with which the reader is familiar.

Active verbs should be used wherever possible. "When silver paint is applied to a water tank more heat is reflected," has far less vitality than "Painting a water tank silver makes it reflect more heat." The writer's goal should be to keep his subject constantly moving, not standing still. Use of the imperative mood—"Examine the meter and proceed . . . ," for example—is also extremely effective. At all times the writer should be dynamic, rather than static—active, rather than passive.

The Seven C's

Everyone knows that the seven seas have long been a means for going places. The following are what might be called the "seven C's" of good writing. If properly applied, they should lead to smooth sailing in the preparation of engineering reports.

To be effective, writing should be: [1] clear, [2] complete, [3] coherent, [4] concise, [5] correct, [6] cordial, and [7] convincing.

Let us briefly examine each of these seven requirements and see how they can help produce good reports.

1. *Clear*. Our major purpose in any written communication is to transmit ideas that are clearly understood. We have already discussed some of the benefits to be gained from using short sentences and paragraphs; from avoiding big words and complex technical terms; from being concrete and not abstract. It is easy to see how active and imperative verbs will add life and interest to a report. Ideally, the material should be presented in such a manner that it can be read and understood as quickly and easily as possible. The reader should understand at once what the situation is or what has happened. The information should be presented in a way that maintains the reader's interest all the way through the report.

2. *Complete*. The report must supply all the data which the reader will require to understand fully the ideas the writer is attempting to convey. An attempt should be made to anticipate every question or objection which might be raised. If at all possible, an acceptable answer should be included in the report before the question ever comes up. There are a number of other small details which contribute to the completeness of the report and therefore increase the chances of a favorable reception. The report should be dated and signed. It should be clear for whom the report has been prepared. Under certain circumstances, it should also be made clear whether a reply is expected and if all action will be held in abeyance until specific instructions are received.

3. *Coherent.* The value of a well planned outline has already been emphasized. Such an outline arranges the various sections of the report into a logical sequence. The several sections must follow a natural progression and should not be mutually exclusive. There should be a definite link between each of the sequential steps through which the reader is led in the orderly pursuit of the writer's intention. All writing represents an attempt to transmit ideas to the reader. Most water works men are familiar with the friction losses that occur when water flows in a pipe. In like manner, when words flow in the channels of written communication an attempt should be made to minimize abrupt changes in direction and other causes of "friction losses." These reduce transmission speed and develop turbulence and eddy currents which distract the reader from the smooth flow of ideas.

4. *Concise.* The people by whom the report is read probably spend a great deal of time reading written material in one form or another. For this reason, there exists a very real obligation to save the reader's time. Furthermore, a report which does not get to the point quickly, which contains superfluous discussion, and which includes many meaningless stock phrases and unnecessary words, is certain to arouse little interest in the busy executive. Often writers tend to tell the reader all they know about the subject rather than just the information he needs to know. Many unessential ideas and elaborations are included that need not be expressed at all. A good report which contains only the essential facts can express much in only a few words.

5. *Correct.* A report is of little value unless it is based on the facts which the writer has been able to determine. If it is only based on an opinion, this should be stated. Care should be taken that any such statements of opinion conform with company policy. Misstatements of fact and policy can be costly and embarrassing. All figures and data should be double-checked for accuracy. The report should also be free from errors of grammar, spelling, and punctuation. Such errors tend to lower the reader's opinion of the worth of the report, as well as disrupt his comprehension of the report's message.

6. *Cordial.* It has already been emphasized that the writer should act as though he were speaking personally to the reader. Conciseness in a report need not make it a cold, unfriendly document. On the other hand, the elimination of the old, familiar, qualifying phrases and clauses demands that particular attention be devoted to making the reader feel that the report was addressed personally to him and in a warm and sincere manner. Care should also be taken to assure that the report is free from words which would needlessly antagonize the reader. There should be no ultimatums, insinuations, or hidden accusations. In certain delicate and touchy situations this may require considerable tact in handling a matter graciously, yet effectively.

7. *Convincing.* If the writer does not sound entirely convinced of the wisdom of his suggestions, there is little likelihood that the reader will be completely satisfied. Writing in a positive tone that evidences a mastery of the subject and an assurance of the soundness of one's recommendations will help sell one's ideas to the reader. Those who read the report will appreciate a firm, convincing attitude on the writer's part. They sincerely hope to

receive a report in which they too can place the utmost confidence.

Form and Arrangement

The form and arrangement of the report itself can do much to improve management's acceptance of it and gain its wholehearted approval. The *title* of the report should be as short as possible and yet be completely descriptive of its purpose. The title should tell the reader as briefly as possible what the report contains.

If the report is at all lengthy, a concise *summary* should be presented at the beginning. Such a summary should indicate the purpose of the study, the pertinent findings, and the major recommendations and conclusions to be drawn from the investigation. The summary is similar to the lead paragraph in a newspaper article. It presents the major facts, but very few details. It arouses the reader's interest and creates a desire to read the rest of the report.

For long engineering reports, a *table of contents* is very helpful. It shows the sequential arrangement of the report. It indicates the specific sections and the subjects discussed. At a glance, it enables the reader to refer directly to those sections in which he is most interested.

A generous number of *charts, tables, drawings,* and other *illustrations* should be used wherever possible. Such visual aids create a change of pace and tend to rejuvenate the reader's interest. They can be effectively used to call attention to the major points the writer wishes to emphasize. When using such pictorial presentations, however, one must be careful not to include so much detail in them that the reader, lacking an engineering or scientific background, tends to become confused.

The proper place for technical details is in an *appendix*. Here are buried the factual data so necessary to substantiate the findings, but so uninteresting to the great majority of those who will read and take action as a result of the report.

Several excellent books have been written on report writing. Two which are particularly helpful in the preparation of engineering reports are those by Kerekes and Winfrey (*3*) and by Nelson (*4*).

Summary

It has been seen that effective engineering reports can be of much value in guiding management toward making sound economic decisions. In planning and writing the report, consideration must always be given to the attitudes, interests, and backgrounds of those for whom it is written. Effective writing should be clear, complete, coherent, concise, correct, cordial and convincing. Attention to the form and arrangement of the report itself promotes its acceptance. The reward for a well prepared engineering report is its enthusiastic approval by top management.

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Effects of Media, Temperature, and Humidity on the Development of Coliform Organisms on Molecular Filter Membranes

——John D. Eye, James G. Gardner, and Jack F. Neel——

A contribution to the Journal by John D. Eye, Assoc. Prof. of Civ. Eng.; and James G. Gardner and Jack F. Neel, Research Assts.; all of the Dept. of Civ. Eng., Virginia Polytechnic Inst., Blacksburg, Va.

THE membrane filter procedure for the bacteriological examination of water, although not formally introduced into the United States until 1947, has been used in Europe for many years. Germany and Russia utilized the membrane filter procedure for the bacteriological control of their water supplies during World War II, when other laboratory facilities were not available. Since 1947, many papers have been written on the applicability and feasibility of using the molecular filter (MF) membrane in sanitary water analysis.

From 1947 through 1954, much research was devoted to finding an acceptable membrane filter procedure for detecting and enumerating coliform organisms in water. The committee which prepared the tenth edition of *Standard Methods* (*1*) recognized the potentialities of the MF membrane procedure and listed it as a tentative standard. Since 1954, several articles have been published which recommend acceptance of the test for the bacteriological control of potable waters.

The procedure recommended in *Standard Methods* employs a two-step liquid nutrient application and careful control of temperature and humidity during incubation. The two-step application requires considerable time for preparation of media and materials and is cumbersome to use because of the necessity of transferring the MF membranes from the enrichment medium to the differential medium at the end of 2 hr. Because of this, efforts have been made to develop a simpler procedure which would yield reliable results. Hajna and Damon (*2*) have reported a single-step procedure utilizing a modified Endo broth which does not require autoclaving or the use of a preliminary enrichment medium. Slanetz and Bartley (*3*) found that an enrichment period was unnecessary and that the filters could be incubated directly on the two-step differentiating medium. Sterilized pads containing dehydrated scheduled nutrient (DSN) Endo are available commercially; these may be used simply by adding sterile water. Goetz and associates (*4*), in comparing the DSN with the two-step procedure, stated: "The dehydrated schedule appears to perform as well or better than the liquid nutrient application."

The need for carefully controlling temperature and humidity has also been questioned. For example, some

investigators (*5, 6*) believe that a dropping temperature such as is obtained in a thermos incubator is advantageous because certain types of organisms will be inhibited and the size of the colonies will be smaller, thus reducing possible interference and confluence. Slanetz and Bartley (*3*) report that a saturated atmosphere is unnecessary because the humidity in an ordinary incubator is sufficient to support growth of coliform organisms on a membrane surface.

The relative simplicity of the MF membrane technique as compared with the standard dilution tube test makes it particularly desirable for field studies and for small water works which do not have elaborate laboratory facilities. Field kits employing the MF membrane are available. The early field kits used thermos bottles for incubating the filtered organisms, whereas recent kits are equipped with battery-heated, constant-temperature incubators.

The object of the investigation reported here is to compare the effect of the type of media, temperature, and humidity on the development of coliform organisms on MF membranes. If these factors are less critical than originally believed, the MF procedure will be more widely acceptable than at present.

In order to assure that an adequate density of coliform organisms would be present in each sample, nonchlorinated well water was inoculated with a small quantity of raw sewage. By the use of several trial-and-error dilutions, the amount of raw sewage required to give positive results and still be less than the maximum reliable coliform-organism count was determined.

After the dilutions were made, the sample was mixed for approximately 5 min before the first 50-ml sample portion was withdrawn. Each time an individual portion was drawn off, a small amount was first wasted in order to insure a uniform inoculum for a given test series. A magnetic stirrer was used to mix the sample.

The comparison of the development of coliform organisms on four media—two-step Endo, Hajna-Damon (H-D) modified Endo, single-step Endo, and DSN Endo—was made in accordance with recommended procedures. Duplicate MF membranes were incubated for each medium.

The study of the effects of dropping temperatures of incubation on the development of coliform organisms was made by using a gallon thermos jug and a quart vacuum bottle as incubators. The thermos incubators were filled with water at 110–115°F at least 30 min prior to use. This gave each container sufficient time to become thoroughly heated. Just prior to placing the duplicate sealed petri dishes in the incubator, the water was adjusted to approximately 98°F. Various rates of temperature drop were obtained by suspending the thermos units outside laboratory windows, by placing them at room temperature, and by putting them in an incubator in which temperatures of 5–40°C could be held constant. Control samples were incubated in a constant-temperature incubator.

All temperatures were recorded by a six-point electronic temperature recorder utilizing thermocouples. The temperature of each unit was printed every 3 min on a strip chart.

The evaluation of the effect of humidity on coliform development was made by incubating at various controlled relative humidities ranging from 13 per cent (which was an average value found in a standard 35°C incu-

bator) to near saturation. The desired level of humidity less than saturation was obtained by exposing a definite surface area of water per unit volume of sealed incubator space to evaporation (*see* Fig. 1). The MF membranes were placed in the bottom section of a pyrex petri dish, covered with a top section, and inverted for incubation.

A saturated atmosphere for incubation of control samples was obtained

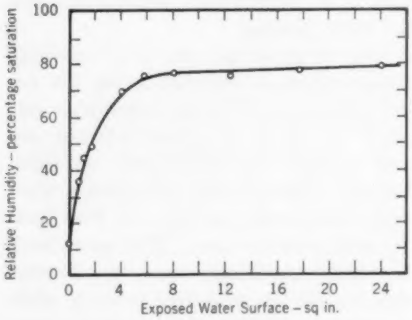

Fig. 1. Relationship Between Relative Humidity and Exposed Water Surface

Desired level of humidity (less than saturation) was obtained by exposing a definite surface area of water per cubic foot of incubator space to evaporation. The constant temperature was 35°C, air circulation by thermal convection current.

both by placing an inverted petri dish bottom 1 in. above a free water surface (water bath) and by sealing a top and bottom section of petri dish together with a 1½-in. section of 1¾-in. diameter Gooch tubing.

Visual comparisons were made of sheen development, inhibitory effects, colony spreading, and coliform-organism counts on the four test media. The filter counts were analyzed by statistical correlation and analysis of variance.

Discussion of Results

Comparison of various media at 35°C. The best sheen was exhibited by the DSN preparation and the poorest by the H-D broth. The single- and two-step Endo broths almost always produced a good sheen. The high quality sheen observed on the DSN was probably the result of the dark, almost black, background displayed by the filter following incubation. To substantiate this theory, several black filters were used with the two-step application. The effect was the same as with the DSN. The black background absorbed the light, whereas the coliform-organism colonies reflected the light, thereby making the sheen more pronounced. The black filters performed as well as the white with respect to the coliform-organism count. Black MF membranes of the type used are somewhat higher in cost and are not susceptible to standard sterilization procedures, therefore only a few were employed in the investigation.

TABLE 1

Coliform-Organism Counts for Various Media at 35°C

Medium	Coliform Organisms per 100 ml
Two-step Endo	172.8
H-D modified Endo	126.9
Single-step Endo	113.6
DSN Endo	113.2

The inhibitory effect of the DSN was excellent. Very few colonies were found growing on the DSN that did not possess the characteristic sheen of coliform organisms. A few colonies of noncoliform organisms grew with no apparent restriction on the two-step and H-D nutrient media. The single-step Endo broth gave from fair to good inhibition.

In a few instances, colony confluence made counting difficult. The spreading was caused principally by the uncurbed growth of noncoliform colonies along the grid lines of the filter. The two-step and H-D applications displayed the most spreading, with the single-step Endo evidencing a lesser degree. At no time did the DSN application exhibit spreading.

To evaluate the media more fully, a comparison was made of the coliform-organism counts. The overall average counts of 396 determinations for the four media are listed in Table 1.

The two-step application recommended in *Standard Methods* gave the highest counts and was used as the basis for a correlation comparison among the remaining media. The correlation coefficients were all within the range of 0.8–0.9, indicating a high degree of correlation.

Comparison of media at dropping temperatures. The effects of dropping incubation temperatures on the various media are shown in Fig. 2. The ratios of the dropping temperature counts to the constant 35°C counts were computed to illustrate the effects of the drop rate. An average ratio for each drop range was calculated to show the performance of each medium for that particular range. It was found that a drop rate of less than 1°F per hour produced counts which were equal to or greater than those obtained by standard 35°C incubation. At a drop rate greater than 2°F per hour, however, very few colonies of coliform organisms developed.

The DSN and single-step Endo mediums demonstrated an ability to support the growth of coliform organisms over a wide range of drop rates. At rates below 2°F per hour, the DSN and single-step Endo gave excellent results. The count on the two-step and H-D applications declined steadily at drop rates greater than 1°F per hour. The DSN was the only medium that supported at least limited growth of coliform organisms over all of the tested ranges of drop.

TABLE 2

*Coliform-Organism Counts for Various Media at Incubation Temperatures of 35°C and 32°C**

35°C	C32°	35°C	32°C
DSN Endo		Two-Step Endo	
63	81	114	187
133	169	189	225
67	74	110	113
27	47	79	88
7	9	15	31
10	13	28	32
73	48	104	100
20	26	42	40
138	152	153	183
26	33	79	93
24	24	41	52
6	11	7	17
Single-Step Endo		H-D Modified Endo	
167	233	170	227
169	225	190	196
40	71	64	94
37	42	23	29
20	40	13	38
21	28	10	25
54	80	71	88
29	53	30	44
67	93	40	67
31	49	31	61
9	9	5	7
8	9	10	12

* Number of coliform organisms per 100 ml of sample.

An evaluation of the characteristics of the coliform organisms under conditions of dropping incubation temperature was made, using standard 35°C as a basis. Somewhat smaller colonies were produced at dropping

temperatures. This was an advantage, as confluence was greatly reduced. With the exception of the H-D broth, there was no apparent difference in sheen. The sheen was improved on the H-D medium.

Standard 35°C incubation compared with 32°C incubation. A study of the results obtained by the dropping incubation temperatures indicated that a lower incubation temperature would produce higher coliform-organism counts than would be obtained at 35°C. For the purpose of this investigation, 32°C was arbitrarily chosen as the test temperature.

The results, as shown in Table 2, indicate that for the samples tested, 32°C yielded higher coliform-organism counts. This explains the ratios greater than 1.00 found when using dropping temperatures of incubation. Further studies are necessary to determine the exact temperature at which the greatest coliform-organism count will occur.

Effect of various levels of humidity on coliform development. By comparing the results of 281 coliform-organism determinations at various levels of constant relative humidity, it was determined that no significant difference in counts occurred at any humidity level above 35 per cent saturation. Data from a limited number of determinations at 35 per cent relative humidity and saturated atmosphere are listed in Table 3. Humidity effects were constant irrespective of the type of medium used.

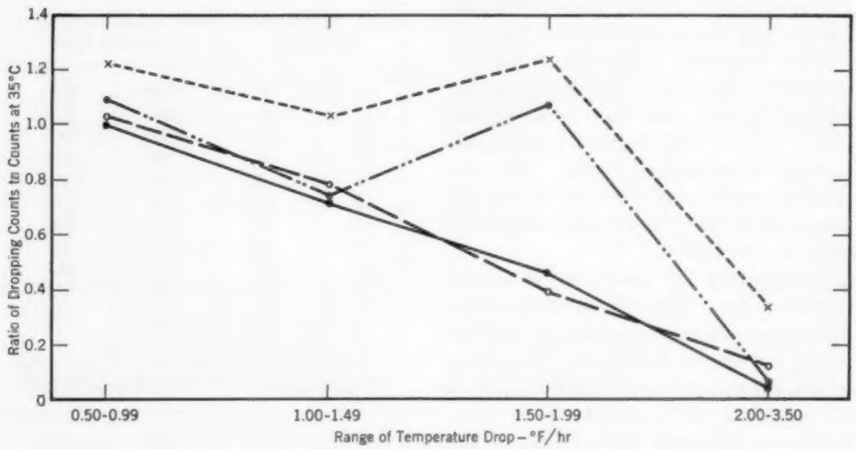

Fig. 2. Effect of Dropping Temperature on Coliform-Organism Counts

Data for four media are shown: × indicates DSN; ○, two step; ⊕, single step; and ●, Hajna-Damon.

An investigation of the effect of very low relative humidity indicated that colony development is unpredictable. Complete dehydration of the membrane and pad assembly occurred frequently at humidity levels below 35 per cent; these levels, therefore, were regarded as unsatisfactory for incubation of coli-

form organisms on MF membranes. When heavy growth was present, the membranes at 35 per cent relative humidity exhibited less confluence of colonies, but there was no apparent difference in sheen development at 35 or 100 per cent.

Comparison of methods for regulating humidity. Because of the size of the hygrometer, it was impossible to measure accurately the relative humidity in the water bath and sealed petri dishes. From observation of the condensate, it appeared that incubation 1 in. above a water surface most nearly approximated 100 per cent relative humidity. Plastic disposable petri dishes were unsatisfactory as a means of incubating in a vacuum bottle because leakage was noted in several dishes when tested under a head of 6 in. of water. Plastic film and Gooch rubber tubing were satisfactory. It is probable that some difficulty may be encountered in the use of plastic film at low temperatures because if applied cold it may expand and lose adhesion when placed in a 35°C incubator. No malfunction was noted in using the Gooch rubber tubing sections at 0°C.

It is possible to maintain a definite level of humidity by exposing a free water surface of a definite area for a unit volume of incubator space (*see* Fig. 1). The desired level was attained about 12 hr after inserting the water container. The normal humidity noted in an unsealed 35°C incubator was 10–15 per cent when the room humidity was 35–50 per cent.

TABLE 3

Coliform-Organism Counts on Various Media at 35 Per Cent Relative Humidity and Saturated Atmosphere *

Two-Step Endo			Single-Step Endo			H-D Modified Endo			DSN Endo		
Water Bath	Sealed	Open	Water Bath	Sealed	Open	Water Bath	Sealed	Open	Water Bath	Sealed	Open
30	—	59	29	—	35	26	—	36	7	—	12
92	128	110	98	106	97	106	110	105	57	112	77
97	99	97	65	60	72	69	56	69	43	65	68
52	55	50	45	32	48	50	41	30	32	35	34
36	35	25	21	16	9	20	5	19	20	13	16
62	90	71	23	15	12	12	17	22	24	17	20
252	255	239	263	206	251	192	149	166	127	76	111
205	213	232	133	145	142	102	108	122	83	113	94
217	230	244	163	134	130	144	87	101	144	133	155

* Number of coliform organisms per 100 ml of sample.

Summary and Conclusions

As the result of the investigations described, it is believed that the MF membrane procedure might be modified to include the following findings:

1. The DSN exhibits the best sheen and inhibitory effects, but the two-step application gives the highest coliform counts.

2. A dark background facilitates counting. The dark background absorbs light, whereas the coliform colonies reflect light, thus giving a more pronounced sheen.

3. Thermos incubation is adequate, provided certain precautions are taken.

The temperature within the thermos should not be allowed to drop more than 1°F per hour.

4. Gooch rubber tubing sections provide the most satisfactory seal on petri dishes for incubation in a thermos jug.

5. For the test sample, 32°C gave higher coliform counts than did the recommended 35°C incubation temperature. Further investigation is necessary to establish the exact temperature of incubation when utilizing the MF membrane.

6. Coliform organisms may be successfully incubated on MF membranes at any level of relative humidity above 35 per cent.

7. The desired level of humidity may be obtained by subjecting a given surface area of water to evaporation with convection air circulation within a sealed incubator.

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Use of Molecular Filter Membranes for Water Plant Control Tests and Clearance of New Pipelines

A. Adler Hirsch

A contribution to the Journal by A. Adler Hirsch, Water Purif. Supt., Dept. of Water & Sewerage, Shreveport, La.

THE molecular filter (MF) membrane procedure for the bacteriologic examination of water, as described in the tenth edition of *Standard Methods* (*1*), offers both reduced incubation time and simplified manipulation as its most significant advantages over the long established dilution techniques employing broths. In addition, it permits the direct count of discrete colonies rather than a statistical indication of the so-called "most probable number" (MPN) of coliform bacteria, and allows the sample volume to be varied almost without limit to suit conditions, without consumption of an extra amount of medium. Differentiation and enumeration of important groups of organisms are often possible directly, without requiring initial enrichment or other preliminaries. Apparatus and techniques are available for various types of usage in the field.

Details of the various MF membrane procedures will not be discussed, as they have been adequately covered by the developers (*2*) and other investigators. Further studies (*3*) have established the utility of the MF membrane and its practical, qualitative working equivalence to the broth methods (*4*).

Disadvantages of the MF membrane are relatively minor, though still real. Highly turbid samples cannot generally be tested. Only a single sample can be processed through a filter assembly at a time, thereby requiring a somewhat longer period to plant a set of samples than is needed with lactose broths and BGB. The filter cylinder and porous plate support must be sterilized between each sample. Initial cost of the filter apparatus exceeds that of the glassware needed for broth tests. Membranes and impregnated pads are decidedly more expensive than the equivalent amounts of broths. Following the *Standard Methods* broth procedure, the author found that the maximum cost per standard sample for the full test was 7.8 cents—2.8 cents for five 30-ml tubes of lactose broth at $1.25 per pound and 5 cents for five 20-ml tubes of BGB at $5.50 per pound. (The cost of the lactose broth was taken for completely negative tests.) In comparison with these costs, the cost of MF membrane and dehydrated-nutrient pad per standard sample was found to be 64 cents. Thus, the MF membrane procedure is 23 times as costly as the lactose broth application alone, and 8 times as costly

as the full lactose-BGB test; that is to say, the MF membrane procedure is 8–23 times as costly as standard broth procedures, depending upon the purity of the water examined, As the MF procedure becomes popular, however, the quantity production of membranes will undoubtedly reduce their cost.

Water Plant Control Tests

The choice between retention of standard broth techniques or adoption of the MF membrane procedure, in addition to the preceding factors, also depends upon the habitual characteristics of the water. This is well illustrated by the following data on the bacterial quality of Shreveport, La., lake water, as determined with standard lactose and BGB tests over the 3-year period 1953–55. Tests of raw water from Cross Lake showed the following results: of 10-ml portions, 97.8 per cent were positive presumptive, 96.8 per cent positive confirmed; of 1-ml portions, 73.7 per cent were positive presumptive, 71.0 per cent positive confirmed; of 0.1-ml portions, 26.0 per cent were positive presumptive, 22.8 per cent positive confirmed. (Raw lake water is almost always positive in 10-ml portions; hence the results are fairly well known in advance.) The MPN counts for this water were as follows: average, 305; maximum, 1,100; minimum, 0.9. The lake water flows through a 3-mile transmission main to the McNeil Street plant after being treated by free residual chlorination, resulting in a raw water which fulfills USPHS standards for drinking water. Of this water, 2.5 per cent of 10-ml portions were positive presumptive, 0.6 per cent positive confirmed. Samples of laboratory tap water from this source were also tested: 0.3 per cent of 10-ml portions were positive presumptive, no portions confirmed. On waters of such uniform characteristics, the accelerated results offered by the MF membrane procedure have no added value for most routine purposes.

As for monitoring during periods of suspected biological sabotage or even gross contamination from sanitary breakdowns, there would probably be little difference between the number of persons contaminated before results were available from the MF membrane procedure and the number contaminated while awaiting the findings of standard broth techniques, as it seems likely that within either period of time almost the entire population served would have consumed the water. Only if the water under suspicion were withheld from use while being tested would the MF membrane prove itself superior by providing an earlier basis for decision regarding bacteriologic safety. Basic study of a more rapid monitoring method employing broth prepared from lactose labeled with the radioactive isotope C^{14} has been described (*5*).

Samples From New Pipelines

For most profitable utilization of the advantages unique to MF membrane bacteriology, the samples should be of unpredictable quality and the need for speedy judgment somewhat acute. Water samples taken on final inspection of new pipelines fit this category well. There is always a rush for tying in new services, regardless of how much ahead of or behind schedule a main-laying project is nearing completion. If a sample is taken in the after-

noon and tested by the MF membrane method, results can be available for decision the next morning, contrasted to the 48–96-hr wait with lactose broth or BGB. The slight extra cost for the filter test is invariably insignificant in comparison with other economic factors and public relations values.

Characteristics of standard broth tests of 25 water samples from new pipelines laid in the Shreveport system in the period May 1955–January 1956 were studied by comparing the distribution of positive and negative 10-ml portions in particular samples. Each sample contained five 10-ml portions. Results are as follows:

1. *Lactose broth test.* In 9 samples, 5 portions were positive presumptive; in 3 samples, less than 5 portions were positive presumptive; and in 13 samples, all portions were negative presumptive.

2. *BGB test.* Of the 9 samples with 5 positive presumptive portions, 8 were positively confirmed, 1 negatively confirmed; of the 3 samples with less than 5 positive presumptive portions, 1 was positively confirmed, 2 were negatively confirmed; and of the 13 samples in which all portions were negative presumptive, 1 was positively confirmed, 12 were negatively confirmed.

It is observed that samples generally are either decidedly contaminated or altogether uncontaminated, the data showing 80 per cent of all samples with either all five presumptive portions positive or all negative.

These samples were also tested in parallel using the MF membrane incubated with moistened Endo and EMB pads. (The latter medium was generally more satisfactory, as it distinguished between *Aer. aerogenes* and *Esch. coli.*) In a few preliminary tests, an inoculum of 10 ml, similar to a standard portion, was used; and later a 25-ml portion was used; but most tests used 50 ml of filtrate in three separate portions of 7, 14, and 29 ml each.

Comparison tests were made with both standard broths and MF membranes (with EMB) on new pipelines. Both 24- and 48-hr determinations were made with the standard broths. The MF membrane tests were made in 18–30-hr periods. Of 25 samples tested, in only one case were the results not in agreement: a sample that had tested positive in both the 48-hr presumptive and 48-hr confirmatory tests tested negative by the MF membrane determination. These data would seem to indicate that in more than 95 per cent of cases both methods give equivalent results on which to base acceptance or rejection of a new pipeline for service to consumers. Economy of time in every instance favored the MF membrane procedure.

In future tests of the MF membrane a full 100-ml sample will be used. The presence of two or more colonies of coliform organisms will cause rejection of the pipeline. A single colony will not bar acceptance, as this is within the USPHS standards limit. If the pad is crowded, the quantitative aspect of the count is of no importance for this particular purpose. The filter cylinder is ringed to mark off this volume without need for other measurement. If an accurate count is desired, the sample can be split into a "geometric" series, such as 33- and 67-ml portions, 14-, 28- and 58-ml portions, and so forth, so that at least one pad will be countable.

In summary, the use of MF membrane bacteriology has been highly accurate and time-saving in clearance testing of new pipelines.

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Discussion—Evaluation of Weather Modification Experiments

Fred W. Decker

A discussion by Fred W. Decker, Asst. Prof., School of Science, Oregon State College, Corvallis, Ore., of the paper, 'Evaluation of Weather Modification Experiments' presented by Frederic A. Berry at the Diamond Jubilee Conference, St. Louis, Mo., and published in the August 1956 Journal, Vol. 48, p. 973.

THE article "Evaluation of Weather Modification Experiments" by Frederic A. Berry (*1*), makes a welcome contribution to clarification of the heated controversy which has raged over the question of the effectiveness of cloud seeding (or "rainmaking" as it is popularly known). Despite all the claims of success by commercial operators selling rain-increase and hail-suppression services, the "practical difficulties" listed by Berry have rendered the effectiveness of such measures debatable. Berry's listing of these difficulties should be reviewed by all readers. His candid description of the uncertainties involved should go far toward returning weather modification to the realm of engineering development from which it was taken by overzealous commercialization.

Unfortunately, Berry did not discuss the reasons for some perplexing uncertainties which will continue to plague this field of development unless sound engineering practices are introduced. Most of the uncertainties in evaluating weather modification efforts arise from the lack of good experimental design. Engineers would ordinarily be aghast at being asked to arrive at such far-reaching conclusions as are involved in weather modification evaluation on the basis of the meager evidence available. Engineers conducting other evaluations are usually able to arrange suitable measurements and a system for the experiment before commencing experimental operations. Such has not been the case in weather modification. In fact, at the Tucson conference on the scientific basis of weather modification evaluation, one point on which the disputants could apparently agree, even though they disagreed on many others, was that the data available for weather modification evaluation were decidedly unsatisfactory. H. C. S. Thom, statistician of the President's Advisory Committee on Weather Control, was reported (*2*) as saying, "We had to resort to commercial cloud-seeding operations because few experiments were being conducted. These weren't scientific experiments and so we worked under a terrific handicap." Confronted by such a situation, statisticians can perhaps more readily determine whether support of a process like rainmaking is an acceptable business risk than whether it is a scientific fact that cloud seeding really worked. This reduces the whole question to a matter of business speculation, rather than scientific proof.

Need for More Data

The great uncertainties in weather modification evaluation arise from the

meager data available and the historical comparisons used. The official climatological network of rain gages is in most places too coarse to sample the nonuniform precipitation pattern in storms having precipitating cells only 1–2 miles in diameter, even within general cloud layers. More rain gage stations are needed. Many AWWA members, incidentally, should be in a position to do something about this for the future. In one Far West project there is only one rain gage per 450 sq miles, which is woefully inadequate for evaluation of the unstable rain showers often treated by weather modifiers, as shown by the Ohio studies of some years ago *(3)*.

If more rain gages are installed, however, the new gage records cannot be used to evaluate seeded storms without new gage records from unseeded storms as well. Historical comparisons, which cannot use new gage records, constitute the second serious cause of uncertainties in weather modification evaluation.

Tree ring studies and the "dry cycles" and "wet cycles" of recent history show that the weather is seldom "normal." Fluctuations appear not only in the records from an individual station but also in the ratio of target to control area precipitation. Similarly, the relationship between precipitation, as determined by the rain gage, and the weather variables of moisture, temperature, and air flow for a group of stations will also vary from year to year. When an equation is determined from historical storms in order to calculate the "natural" amount of target precipitation for a given storm, there is no guarantee that it is valid for the particular storm in question. If the actual rainfall is more or less than the computed amount, it must be recognized that natural variation could have caused the difference.

Statistical Technique

In Berry's examples a "statistically significant" amount of precipitation is one which exceeds 90 per cent of the plotted cases in history. Hence, if the probability is only 10 per cent or less that the yield from a given storm would be exceeded in the target area, the storm will be accepted as having been increased by cloud seeding. Such a level of acceptance has its hazards, however. Concerning a standard of acceptance similar to this, the California State Water Resources Board observed (4) that "even if the true effect is sizably negative there is a good chance of announcing a positive result." Nevertheless, establishing too stringent an acceptance standard will increase the risk of ignoring a genuine effect.

The use of different standards of rarity required for "statistical significance" contributed to the apparently contradictory conclusions announced by two different groups evaluating the same cloud-seeding project *(5–7)*. To label a result as "statistically significant" is not sufficient unless the level of acceptance is also stated.

In Berry's examples apparently only positive effects were considered, and the possibility of "significant" negative departures was ignored. This might not be a safe omission, for the possibility of significantly *decreasing* precipitation by cloud seeding requires more definitive investigation.

A different type of weather modification evaluation not mentioned in Berry's paper, which deals mainly with statistical analyses of overall effects,

would involve a more detailed measurement of the existing cloud and air mass moisture, the actual vertical mixing rate of the seeding agent, and the concentration and threshold activity temperature of the seeding agent after it has finally reached various levels in the cloud. From data such as these it would be possible to compute the amount of precipitation which should fall if seeding were perfectly effective. Before groups or individuals embark upon expensive programs of cloud seeding, it might be wise to compute analytically the maximum effect which can be expected with perfectly effective cloud seeding and then to relate it to the cost. Some meteorologists suggest that perhaps the required conditions favorable for even the modest increases now being suggested would occur so rarely that cloud seeding may not be economically sound, even if it should eventually be proved to produce genuine results.

Controlled Experimentation

Some critics at the Tucson meeting condemned in the most severe terms the kind of historical comparisons described in Berry's paper. Perhaps the controversy has its roots in the implication that such comparisons, linked with the definition of statistical significance as the amount which exceeds 90 per cent of the cases in history, can produce "scientific proof" that cloud seeding succeeded. The historical observations, however, were not originally produced for the purpose to which the evaluators are putting them, the gages are too far apart, and climatic cycles do occur. Everyone, therefore, should be able to agree that statistical analysis of such data may help to decide about further support for cloud seeding as a speculation but cannot readily provide proof that seeding really was effective. Instead, statistical considerations should enter into the planning of experiments which will produce more convincing data. These experiments can contribute new knowledge and can lead toward improvement of weather modification. Whether certain past commercial weather modification projects have been slightly successful is really beside the point. Water users of the West may not be satisfied to invest funds in gross operations lacking experimental design if the results are at best simply gambling odds, especially when it would have been possible at comparable cost to obtain conclusive engineering experiments.

Laboratory experiments in the cold box certainly call for further engineering research to determine the greatest effect that can be obtained from cloud seeding, and the cost thereof. The alchemist might have been satisfied with apparent results without knowing how they came about, but the modern engineer wants to know exactly what the results are and how they can be obtained most efficiently.

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Dollar Value of Fluorides to Children

In the July 1956 *Journal of the American Dental Association,* on p. 38, an article by W. O. Young and W. J. Pelton entitled "Planning a Dental Prepayment Program for Children in an Area of Low Caries Prevalence" compares dental care costs for children in the towns of Nampa and Coeur d'Alene, Idaho. The natural fluoride content of the Nampa water supply was 1.5 ppm; the Coeur d'Alene supply was free of fluorides. In 1951, detailed dental examinations were made of 850 continuous-resident school children in the first seven grades of these communities. Nampa children averaged only 0.35 DMF (decayed, missing, filled) teeth at age 7, and slightly less than two DMF teeth at age 13. Coeur d'Alene children, however, averaged almost two DMF teeth at age 7, almost ten at age 13.

By means of average local dental rate schedules, the cost of total dental care (for permanent and posterior deciduous teeth) was estimated for the children examined (*see* Table).

The table indicates that the average cost of total dental care per child in Nampa for the 8-year period between ages six and thirteen is $284.83 less than that for a child in Coeur d'Alene. As not all of the children in either community received total dental care, it is interesting to note that the treatment actually received cost on the average of $50.07 per child (for the 8-year period) in Nampa, and $180.09 per child in Coeur d'Alene. Although the money actually spent for dental care is lower than the cost of total dental care, the relationship between the two sets of figures is surprisingly close, being 1:3.95 for total care and 1:3.62 for actual care.

Age	Cost of Total Dental Care		Difference in Costs
	Nampa	Coeur d'Alene	
6	$ 9.49	$ 28.58	$ 19.09
7	11.59	37.81	26.22
8	15.75	50.14	34.39
9	15.10	52.61	37.51
10	16.17	50.27	34.10
11	8.94	51.09	42.15
12	10.25	47.19	36.94
13	11.11	65.54	54.43
Total	$98.40	$383.23	$284.83

The authors in part conclude:

> The analysis of data derived from dental examinations . . . indicates that considerable reductions in treatment costs and therefore a corresponding decrease in premium rates may be expected in a community such as Nampa, where the drinking water contains an optimum amount of fluorides. A proposal has been outlined for a prepayment program in a fluoride community, and it is estimated that complete operative care for elementary-school children could be purchased for as little as $15 a year.

The authors also estimate that, to purchase prepayment dental health care for a child in Coeur d'Alene, the premiums would approach $40 a year—an amount which might deter parents from following such a program.

Joints for Steel Water Pipe

Walter H. Cates

A contribution to the Journal by Walter H. Cates, Mgr., Hydraulics Div., Consolidated Western Steel Div., United States Steel Corp., Los Angeles, Calif.

A RECENT inquiry received by the Association from the US Army Corps of Engineers raised certain questions regarding the application of lining and interior coating materials to field joints of steel water pipe with diameters of 12–24 in. Some answers to these questions will be summarized in this article.

Types of Field Joints

Figure 1 shows a "welded slip" joint, as used by the Los Angeles Department of Water and Power for pipe of 21-in. diameter or greater. This type of joint is used widely in the West because of its simplicity, flexibility, watertightness, and strength. After the joint has been welded, the inside can be lined with coal-tar enamel or cement-mortar from within the pipe. For pipe sizes of 21-in. diameter and smaller, therefore, use of the welded slip joint will cause difficulties in lining. One method of lining pipes of smaller size with coal-tar enamel or cement-mortar after completion of the welding is to brush on the lining through a 4-in. coupling which has been shop welded to the pipe close to each field joint. After the lining has been completed, the coupling is closed by a screwed plug.

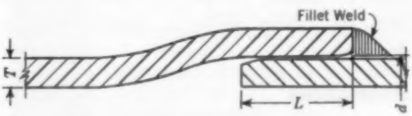

Fig. 1. Welded Slip Joint

For single-welded joints, the length of lap, L, *is 1½ in. For double-welded joints,* L *should be 5 times the plate thickness,* T, *or a minimum of 1½ in. Inside diameter,* d, *of bell should be* $\frac{1}{32}$–$\frac{1}{16}$ *in. greater than the OD of plain end.*

Figure 2 shows a "reinforced expanded bell" field joint, as used at Los Angeles for all pipe of 12-in. diameter or greater. This joint is cement calked in the same manner as a cast-iron pipe bell-and-spigot joint. It can be used for working pressures up to 200 psi. The advantages of this joint are its simplicity and low cost of laying in the field, and the fact that the inside lining remains intact and unaffected by the laying operations.

Figure 3 shows a "bell-and-spigot for rubber gasket seal" joint, which is designed to give a very flexible, watertight, and low-cost field joint without any field welding or damage to the inside lining. It will permit deflections in alignment up to at least a 4-deg angle and longitudinal slippage of at

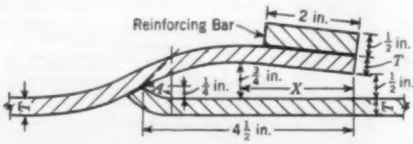

Fig. 2. Reinforced Expanded Bell

T *is the plate thickness. Angle* A *should be 45 deg and distance* x *not greater than* $2\frac{1}{16}$ *in.*

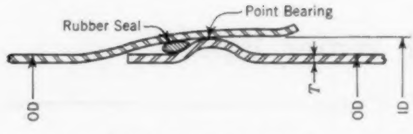

Fig. 3. Bell-and-Spigot for Rubber Gasket Seal Joint

T *is the plate thickness. The rubber seal indicated is an* O *ring, or continuous-ring gasket, as specified in AWWA C300.*

least 1 in. without any signs of leakage. The shoulder on the spigot end acts as a stop for proper joint insertion and self-centering. The greater the water pressure, the tighter the joint will become, and, because the O ring rubber gasket (*1*) is confined by metal on three sides only, the gasket is able to adjust itself to changing conditions or variations that may occur in the joint. The fourth side of the gasket is confined by water.

Figure 4 shows another type of rubber gasket joint used on the West Coast. This joint is easy to install without calking, bolting, or welding. The inside lining will not be affected by the laying operations and any type of lining can be applied in the shop.

Figure 5 shows a mechanical coupling used extensively throughout the United States for effective connection of plain-end steel pipe sections without damage to the inside lining of the joint. This type of coupling provides

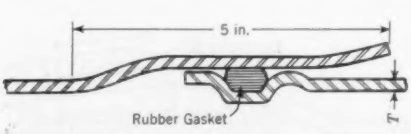

Fig. 4. Rubber Gasket Joint

The rubber gasket shown is an O-ring or continuous-ring gasket, as specified in AWWA C300.

ease of installation, watertightness, and flexibility.

Inspection of Joint Interiors

For pipe larger than 21 in. in diameter, inspection can be made readily by entrance into the pipe. For pipe of smaller size, inspection can be accomplished by observation through a 4-in. coupling attached to the pipe adjacent to the joint, as mentioned above. A new and unique method of photographing casing or pipe interiors has been

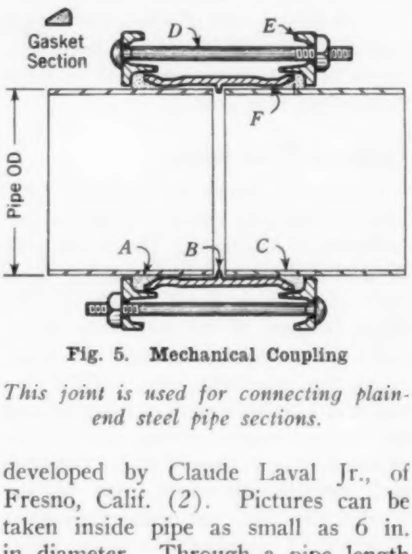

Fig. 5. Mechanical Coupling

This joint is used for connecting plain-end steel pipe sections.

developed by Claude Laval Jr., of Fresno, Calif. (*2*). Pictures can be taken inside pipe as small as 6 in. in diameter. Through a pipe length run of about 2,500 ft, 350 pictures can be taken with a single camera loading.

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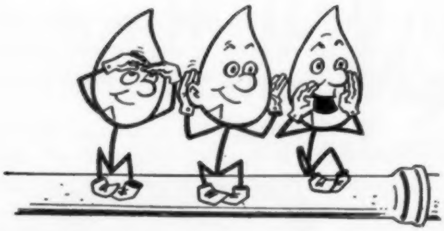

Percolation and Runoff

The Federal Pollution Control Act has been amended to broaden and strengthen its provisions with regard to research, state program grants, and federal enforcement on interstate streams. The law as amended also authorizes the surgeon general to make grants to municipalities for sewage works design and construction. The AWWA Board of Directors, in a resolution on May 11, had expressed strong support for the bill in general but had opposed the sewage works grants-in-aid feature, on the grounds that:

> The financial condition of our cities is such as to make the sale of bonds to build sewage works feasible. The possibility of grants-in-aid for sewage works construction will not only cause delay in construction of such works but may, by creating hopes for similar grants for water works, needlessly delay water works construction fully feasible under normal conditions of financing.

In signing the bill, President Eisenhower pointed out that the grants provision went beyond the recommendations of the Administration, and he urged:

> . . . that no community with sufficient resources to construct a needed sewage treatment project without federal aid postpone that construction simply because of the prospect of a possible federal grant. It should be clearly understood that federal aid will not be available to all communities and, with respect to any one project, the federal funds are limited in amount under the provisions of the bill.

AWWA's research promotion sounds all too characteristic of just plain promotion when we note that it has been campaigning for the development of internal coating and, then, the introduction of alcohol into water supplies. But this is no hangover preparation and prosecution.

The internal coating interest is one that has now been supported by funds from the National Institutes of Health, which has approved a 2-year study of the building up of protective coatings in water distribution systems through water treatment, to be carried on at Michigan State University under the direction of civil engineering professor Robert F. McCauley. Dr. McCauley and his staff will have the counsel of an AWWA advisory committee in conducting the project, which is aimed at learning "how internal coatings can be uniformly laid down in a dense impermeable form which provides high anticorrosion protection without damage to hot and cold water distribution systems." The study will differ from

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previous work in that the emphasis will be on the formation and growth of the crystalline materials which form the coating scales. It is unlikely that Dr. McCauley will explore the effectiveness of milk coatings, at least during working hours.

The alcohol in which AWWA's promoters have become so interested is cetyl alcohol, more morally known as hexadecanol, a chemical which offers great promise as a means of controlling evaporation in water supply reservoirs. With AWWA, in sponsoring research on evaporation control, are a number of other interested parties, principally water utilities in Texas and Oklahoma, working in cooperation with a number of government agencies, including the Public Health Service, Bureau of Reclamation, and Geological Survey. Hexadecanol has been found the most promising chemical of those tested in that it spreads readily on water to form a continuous monomolecular film that permits oxygen and other gases to pass, but provides a barrier against evaporation. Preliminary pan tests by the Bureau of Reclamation have shown reductions of as much as 64 per cent in evaporation. And this summer, the chemical was given a reservoir-scale test on the waters of Oklahoma City's 5.8-acre Kid's Lake by a team of North Texas State College students. Part of this test was the determination of the toxicity of hexadecanol—not its intoxicity, though—that has already been proved nauseatingly nil.

Next project to be pushed, we hope, is the effect of various types of supplies, subjected to various types of treatment, upon the taste of various types of bourbon under various types of physical and mental conditions. Under the scientific aura of the National Institutes of Health, we, ourself, might be persuaded to, at least, participate.

W. A. Kunigk has retired as superintendent and chief engineer of the Tacoma, Wash., Water Div., a post he held for 39 years. Noted for his outstanding leadership in the development of the city's water system, Mr. Kunigk was the first to receive the Fuller Award from the Pacific Northwest Section, which he served as chairman and director. He is succeeded as superintendent by J. A. Kuehl, formerly assistant superintendent.

Fire Prevention Week this year—Oct. 7–13—will commemorate the Chicago fire on its 85th anniversary and the San Francisco fire on its 50th. Hardly a celebration, though, the commemoration will be dedicated to promoting a reduction of the 1,200 home fires, 116 store fires, 105 factory fires, 107 barn fires, 14 school fires, and 4 hospital fires that occur every day. To this effort every water utility can lend a hand on more than philanthropic grounds, for every fire prevented is a lot of water saved—water that is unmetered, unbilled, and unwelcome in the aftermath (not to mention in the New York City subways).

Public-servicewise and public-relationswise, both, Fire Prevention Week offers an opportunity to sell the role of water as a fire fighter, so you'll do well to pour it on beginning Oct. 7.

Thorndike Saville, dean of New York University's Engineering College, has been appointed New York State representative on the Delaware River Basin Advisory Committee, a four-state organization set up last year.

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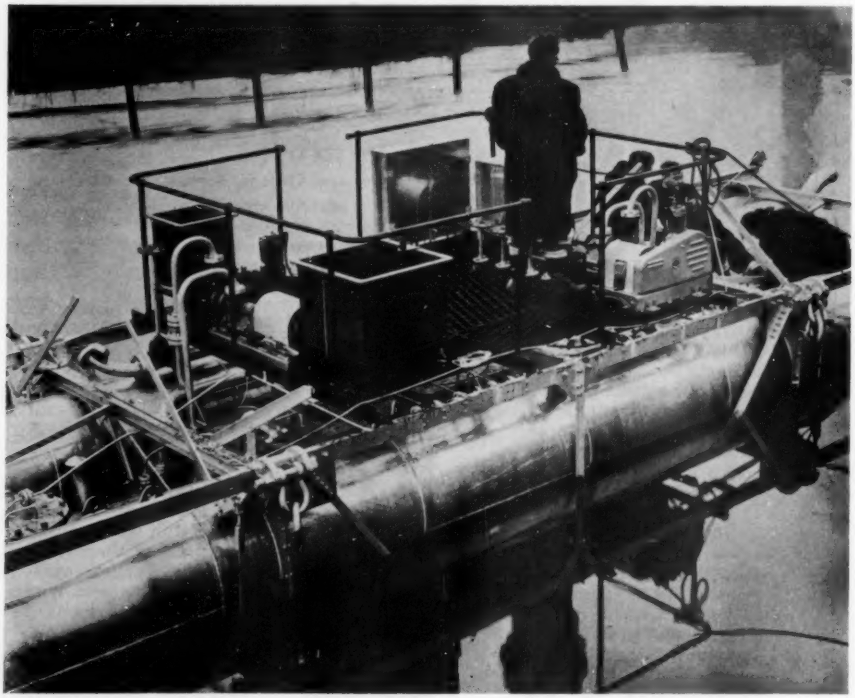
In its element right up to the scuppers is the water works pictured below—a floating version of the landlubberly facilities with which you serve your own community. Produced by the British firm, Marston Excelsior Ltd., for the Iraq government, this water-borne water bearer plies the Iraqian streams, purifying and pumping ashore 2,400 gal (Imp.) per hour where it is needed. And were it to work itself into the ground or were it needed where it could neither drift nor sail nor be towed nor rowed, the entire plant is quickly dismantlable for easy overland transport. In short, with one of these and ambition, a young man could become a real missionary, operating a roving water district for the un-public-water-supplied—one of these, ambition, and some of that elemental scupper stuff, of course.

San Bernardino County, Calif., announces a number of vacancies in engineering positions covered by civil service. Full details can be obtained by writing to San Bernardino County Dept. of Civil Service & Personnel, 236—3rd St., San Bernardino, Calif.

Fischer & Porter Co., Hatboro, Pa., has renamed its Chlorination Div. "Water & Waste Div.," Robert L. Rice, general manager. James F. Haskett has been appointed assistant general manager and John J. Fetch, production manager.



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ASEIB—American Sanitary Engineering Intersociety Board—has begun receiving applications for certificates of special knowledge in sanitary engineering. All certified persons will be carried on a roster to be known as the American Academy of Sanitary Engineering. The basic requirements are registration as a professional engineer, graduation from a college of engineering, and at least 8 years of sanitary engineering experience. Education, experience, and the results of an oral and written examination will determine eligibility for certification. Applicants with unusually high qualifications and at least 15 years of experience may be considered without examination if they file before Jul. 1, 1957. All must pay an application fee of $10 and an examination fee of $25. The intersociety board, which is sponsored by APHA, ASCE, AWWA, FSIWA, and the American Society of Engineering Education, is located at 33 W. 39th St., New York 18, N.Y.

Edward R. Stapley, dean emeritus, Oklahoma Institute of Technology, Oklahoma A&M, Stillwater, took office as president of the National Council of State Boards of Engineering Examiners for 1956–57, following the organization's annual meeting at Los Angeles, Aug. 23–25.

A 34,000-acre summer camp for 30 boys is what the Newark, N.J., water department provided for 8 weeks of last summer on its Pequannock watershed. And the boys—all juniors and seniors in Newark high schools se-

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lected on the basis of their academic record and interest—were provided not only board and room, supervised recreation, television, boating, and fishing, but $1 an hour for the 40 hours a week that they worked at clearing brush, pruning and trimming trees, and, in general, catching up with the maintenance of the watershed area, long neglected for lack of help. Approached as an experiment by the city and the board of education, the project has received the enthusiastic approval of all concerned: the boys, who netted $320 for the season, plus a tan that was the envy of the neighborhood when they sported it at home during weekends; the water department, which got "more than its money's worth" in work, not to mention public relations; and the community as a whole, which found a way to provide an interesting and useful outlet for the energies and enthusiasms of its teen-agers. Next year it is to be more teen-agers in more departments in and around Newark, but we'd guess that the water department would be a pretty general first choice—an advantage, by the way, that might be capitalized by other water utilities.

What price teen-agers—not only as water workers, but as future customers?

Gerard J. Dierker has been named to the newly created post of executive vice-president of Jamaica Water Supply Co., Jamaica, N.Y. He will continue as controller of the company.

Sakuma Dam, the largest construction project ever undertaken in Japan, has gone into full operation. Located on the Tenryu River about 100 miles from Tokyo, the $100,000,000 multipurpose installation can generate 350,000 kw and store over a billion cubic feet of water.

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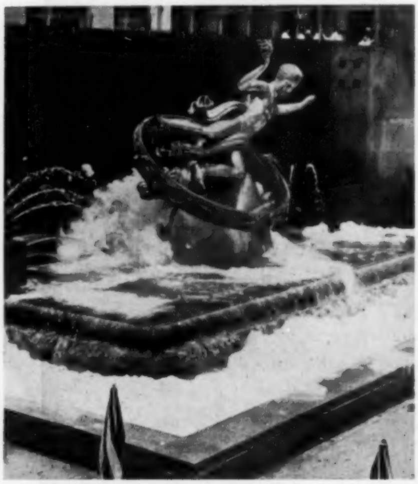
Prometheus bounding—out of a bubble bath—was the picture at the famous Rockefeller Plaza fountain in New York City last July, when someone who may have heard of Atlanta's recent upbubblement (*see* July P&R p. 44) got the urge to deterge what critics have called "Leaping Looie." Apparently a considerable quantity of detergent was poured into the fountain before it was turned on at 7 AM, so that by the time it could be shut



Wide World Photos

off, all of Radio City was bubbly. Scrubbed down, the fountain was in operation again by noon—with a real gleam in Prometheus' eye. Anyway, it was good, clean fun!

Armco Steel Corp. has received a conservation award from the Allegheny County Sportsmen's League in recognition of the pollution control activities of the company's Butler, Pa., works. The citation states that the plant has for many years followed the practice of neutralizing its liquid wastes and that, when accidental pollution has occurred, Armco "has voluntarily made reparations far beyond the requirements of law for stream damage and losses to aquatic life."

Fred D. Ordway Jr. has been appointed director of the Portland Cement Assn. Fellowship at the National Bureau of Standards in Washington, D.C. The fellowship, which operates as an integral section of the association's research department, employs a staff of scientists in the Bureau of Standards laboratories to conduct basic research on portland cement. Dr. Ordway has been acting director since the retirement of Dr. R. H. Bogue in 1954.

A traineeship program for graduate public health personnel is provided for in the Health Amendments Act of 1956, recently passed by the House and Senate. In order to get the program under way as early as possible, priority will be given to individuals who are newly entering or presently engaged in public health work. Information on the program will be available from the Div. of General Health Services, US Public Health Service, Washington, D.C.

Unheard, but not unheard of, is the application of noiseless noise to the removal of undirtless dirt from almost anything. As a matter of fact, the idea of vibrating the impurities right out of water was a pet program of ours until the development of atomic energy gave us gamma rays to play with. But even if water disinfection by gamma irradiation soon proves to be practical, soundless sound isn't likely to be stilled. One of the many new applications under development is the

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ultrasonic cleaning of water meter parts, a procedure that was discussed and demonstrated last April at the Utica meeting of the New York Section by George Lobb, a representative of the Bendix Aviation Corp., Teterboro, N.J. Although Mr. Lobb's references to such frightening things as "implosions" and "piezoelectric and magnetostrictive transducers" was a little higher pitched than our ears, his points that ultrasonic cleaning made it possible to get meter parts cleaner and to do the job with a minimum of meter disassembly were easy enough to understand. And having been convinced that ultrasonic vibrations have no trouble in reaching out-of-the-way places, that they can remove even insoluble dirt, and that they are safe to use even on the most delicate instruments, we're all set to try some ourself next Saturday night. Baths are for the birds, we say; this really sounds sound!

The national advertising campaign of Cast Iron Pipe Research Assn. was recently named a winner in the *Saturday Review's* fourth annual contest for distinguished advertising in the public interest. More than 400 campaigns were screened by the weekly literary magazine's panel to determine the 25 awardees. The association's advertisements have stressed the need for long-range planning and realistic rates in the water industry, and have urged public cooperation and support. The campaign was prepared by the H. B. Humphrey, Alley & Richards agency. Other award winners included American Cyanamid, Caterpillar Tractor, and General Electric.

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"Tank trucks for Texas" was an idea by which we tried, 2 or 3 years ago, to dramatize the king-size drought that was being experienced in the Southwest. And now, with water shortages still plaguing the area, it looks as if at least the first act of the drama is actually being performed. The scene, as shown above, is a front lawn of a lovely home in Terrell, Tex. The players—not with the truck—are two of Terrell's belles of the future. The plot, as revealed in a classified ad in the Terrell *Tribune,* reads:

> $10.00 WILL deliver 1,000 gallons clear rain water leaving truck your premises overnight for unloading through garden hose your plants. Jim Breeden.

The hero, as far as we're concerned, is author-producer Breeden, not only for knowing what water is worth, but for getting that much for it. And if watering water rates a penny a gallon, just imagine what washing water or cooking water or drinking water would bring. But that will be Acts II and III.

Meanwhile, we can't help wonder where or how friend Breeden gets his rain water when he needs it. After all, it isn't supposed to have rained very much. But then, what water wasn't rain water once? And who, for that matter, cares?

Texas, anyone?

James H. Stephens, formerly chief sanitary engineer of the South Carolina Board of Health, is now associated with Harwood Beebe Co., municipal and civil engineers, Spartanburg, S.C.

George G. Schmid, engineer-manager of the Southeastern Oakland County (Mich.) Water Authority, has been named director of public service at Grand Rapids, Mich.

Sidewalk superintendents, like back seat drivers, should normally be heard but not listened to. On occasion, though, it does pay to be a little more than just polite, as a metropolitan area water main installation crew in East Hartford, Conn., discovered last month when it continued excavations for a new line despite strong protests from the gallery that the job had been done the year before. A look at the orders *after* the year-old main had been uncovered disclosed the fact that the crew had picked the right street all right, but the wrong town. Ah well, it isn't the first time or the worst time a contractor has ignored sidewalk suggestions. The one who built the tower at Pisa wouldn't listen either!

R. W. Schneider has been appointed field sales engineer for Indiana, Michigan, and Ohio by Infilco Inc. He replaces J. P. Manger, who will work in sales project engineering at the company's Tucson, Ariz., headquarters.

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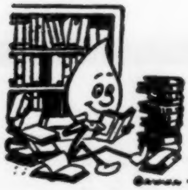
Albert L. Morris has been named to the new post of director of company relations at Dorr-Oliver Inc., Stamford, Conn. With the retirement of Harold Oliver as vice-president for public relations and personnel, Mr. Morris has assumed overall responsibility for all public relations and personnel functions of the company.

A paternity suit was narrowly avoided—by two months, that is—last May when the Cleveland Water Department was advised by an irate female resident that the fluorides in the city's water supply had made her pregnant. As Cleveland's fluoridation program didn't get under way until Jul. 2, Water Commissioner Frank Schwemler felt pretty sure that he could prove the department's May impotence. The accusation, though, may be the first complaint against fluoridation that isn't given the full treatment by antifluoridationists—not because it is any more illogical than some of the others, but because the subject is one on which the least informed people seem to have more than adequate information. On the other hand, we wouldn't want to bet that the claim wouldn't make telling propaganda in some quarters.

At any rate, when Fluoride "Smith" Jr. arrives next February, he can be a lot surer of good teeth than parentage.

Albert L. Blackwell, paving and public works engineer for Portland Cement Assn. in northern New Jersey, has been appointed to the New Jersey Board of Professional Engineers & Land Surveyors.

(*Continued on page 90 P&R*)



The Reading Meter

Flood. *David Dempsey. Ballantine Books, 101 Fifth Ave., New York 3, N.Y. (1956) 138 pp.; paperbound, 35¢; hardbound, $2*

Flood is a story of water rather than water supply—a foot-by-foot report of the drowning of a good portion of Connecticut and Pennsylvania last Aug. 19. It is a gripping story because it is told in terms of people in peril, measuring the impact of the flood in terms of their actions and emotions. And it is well told, not only because the author is competent but because he has been conscientious enough to dig out his information directly from the people and places involved. Although the problems of water supply and of other phases of sanitary engineering are brought in only incidentally, the overall picture it gives of what an emergency can involve makes the book well worth reading. And water works men will recognize in the concluding chapter, "Politics of Disaster," many of the familiar frustrations in getting action once emergencies are over.

Municipal Public Works Cost Accounting Manual. *Walter O. Harris. Public Administration Service, 1313 E. 60th St., Chicago 37, Ill. (1955) 98 pp.; paperbound; $3*

This manual has been prepared by the author, with the assistance of other PAS staff members, for the guidance of municipal officials in installing and maintaining a cost-accounting system. The system outlined is complete with respect to classifications, forms, records, procedures, and cost reports and statements. Designed primarily for hand bookkeeping, the system can be adapted for mechanical operation.

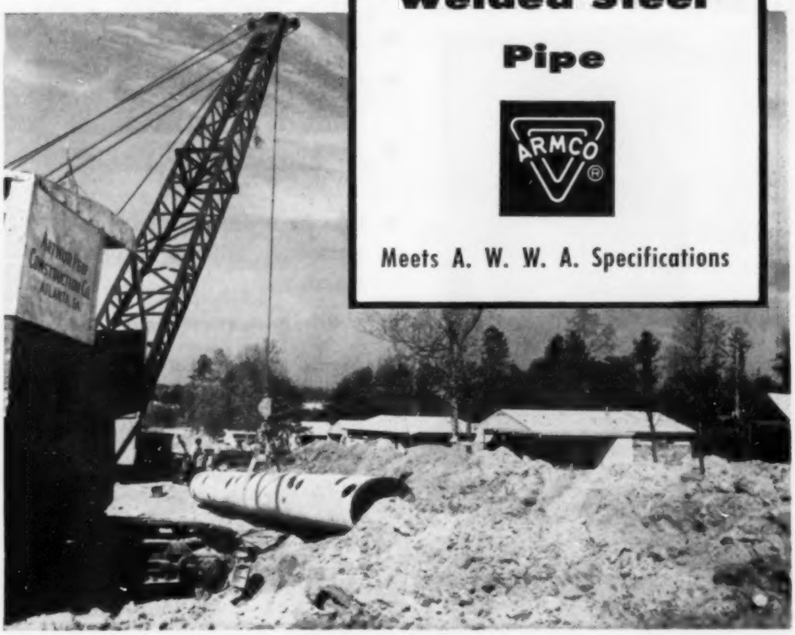
Although the manual is broader in scope than is necessary for the water works man, some of the material presented should prove of great value to operators of smaller plants, as well as to public works officials whose functions include water utility operation. The establishment of units of measurement is particularly commendable. A number of the record forms are suitable for water works use as given or with modifications.

Clarification, Sedimentation, and Thickening Equipment—A Patent Review. *William L. Barham, Joseph L. Matherne & Arthur G. Keller. Bulletin No. 54, Engineering Experiment Station, Louisiana State University, Baton Rouge, La. (1956) 208 pp.; paperbound; $2*

The authors have made a survey of patents in the period 1849–1954 covering primary and auxiliary devices for the clarification of liquids. Several hundred patents are chronologically listed and discussed, with illustrations.

Annotated Bibliography on Hydrology (1951–54) and Sedimentation (1950–54), United States and Canada. *Joint Hydrology-Sedimentation Bulletin No. 7, Subcommittees on Hydrology and Sedimentation, Inter-Agency Committee on Water Resources. Government Printing Office, Washington 25, D.C. (1955) 207 pp.; paperbound; $1.25*

This publication, prepared by the American Geophysical Union, brings up to 1954 its earlier bibliographies on these subjects. According to the preface, every effort has been made to cover all the literature published in the United States and Canada. There is both an author and a combined subject–place names index.



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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

CHEMICAL ANALYSIS

The Determination of Argon in Natural Waters With Special Reference to the Metabolisms of Oxygen and Nitrogen. K. SUGAWARA & I. TOCHIKUBO. J. Earth Sci. Nagoya Univ. (Japan), **3**:77 ('55). Argon is detd. as residual gas after treatment with Ca to fix N. Anal. given of 8 wells, 3 lakes, and 1 rain water for O, N, and A. Some of waters are undersatd. with O and supersatd. with N.—*CA*

Simultaneous Spectrophotometric Determination of Calcium and Magnesium. A. YOUNG, T. R. SWEET & B. B. BAKER. Analyst. Chem., **27**:356 ('55). Rapid and accurate spectrophotometric method has been developed for simultaneous detn. of calcium and magnesium in water. Method is based on absorption measurements at wavelength of 630 mμ and at pH values of 9.5 and 11.7. Eriochrome black T is used as indicator. From anals. of different mixtures it was found that avg. absolute error was 0.12 ppm for calcium and 0.09 ppm for magnesium. —*WPA*

Calcium and Magnesium Determination With Sodium Ethylenediaminetetraacetate With Special Regard to Mineral Waters. K. E. QUENTIN. Z. Lebensm.-Untersuch. u. -Forsch. (Ger.), **102**:106 ('55). Literature on method is reviewed and discussed. On basis of information, details are presented for application of method to drinking waters, and 18 waters were analyzed. 94 references. —*CA*

Colorimetric Determination of Chloride Ion Via Ion Exchange. J. L. LAMBERT & S. K. YASUDA. Analyt. Chem., **27**:444 ('55). In colorimetric method for detn. of chloride, the soln. is passed through column contg. granular silver iodate and released iodate reacts with cadmium iodide-linear starch reagent to form blue linear starch-triiodide ion complex. Absorbancy of complex at wavelength of 615 mμ is proportional to original concn. of chloride. No interference is caused by usual concn. of ions commonly found in natural waters. Bromide and iodide ions react in same way as chloride, but are not usually present.—*WPA*

Determination of Chloride and Bromide in Mixtures of Halides. C. MAHR & H. OTTERBEIN. Angew. Chem. (Germany), **66**:636 ('54). Authors describe advantages of using ceric sulfate instead of chromate for selective oxidation of bromide when Cl is to be detd. in solns. contg. bromide and chloride. By adding cerium salt and passing steam through acid soln. Br is practically completely driven off while chloride is not oxidized. If, however, concn. of acid is high, oxidation of chloride is also rapid, and if amt. of acid is low, oxidation of bromide is slow. Authors recommend that in some circumstances Br should not be completely driven off; small residual amt. is recognizable in first potential rise. Iodide must be removed by use of nitrite before addn. of ceric sulfate. Cyanide must be removed by weak acidification and steam distiln. Losses of Cl may also be caused by presence of thiocyanate with bromide or iodide. Method is rapid, and small variations in procedure do not affect it. Expts. were therefore made to combine detn. of chloride and bromide, to extend range of concn. in which method could be used, and to make it usable in presence of iodide, cyanide, and thiocyanate. To avoid formation of I bromide, oxidation of iodide with nitrate is carried out in presence of excess cyanide, forming iodine cyanide which is removed, after destruction of excess nitrate with amidosulphonic acid, by steam distn. Thiocyanate can be destroyed with

(*Continued on page 64 P&R*)

(*Continued from page 62 P&R*)

alk. H peroxide without affecting Cl or Br. Authors describe app. used and give details of procedure for mixts. of chloride and bromide alone and in presence of cyanide, iodide, and cyanide and iodide, cyanide, and thiocyanate. Other ions tested did not interfere, although presence of other salts in very high concns. may cause loss of Cl as hydrogen chloride. Fe ions may interfere in presence of cyanide. Accuracy in samples where ratio of chloride to bromide varied from 100:1 to 1:55 varied from −0.6 to +0.5%. —*WPA*

New Knowledge in the Chlorination of Water and Sewage and on the Determination of Free Chlorine. G. Gad & M. Manthey. Vom Wasser, 20:185 ('53). Authors give brief summary of difficulties and disadvantages of various known methods of detng. concn. of free chlorine in water. Method previously described by them, using sensitive dyestuff, Chlorindikator I, is suitable for concns. of 0.3–0.2 mg/l Cl. Combination of this indicator with soln. of arsenite (Chlorindikator II) has now been developed which is suitable for detn. of 1–50 mg/l. Using this indicator and Winkler's methyl red process, comparisons have been made which give information on chem. processes in breakpoint chlorination of artificially prepared water and of sewage. Comparison of results of 2 methods show that Cl up to breakpoint is mainly in form of chloramine. On addn. of Cl to water contg. ammonia, monochloramine is first formed; as addn. continues, dichloramine is formed, and when about half of chloramine present is in form of dichloramine, this reacts with monochloramine, and N and hydrochloric acid are formed. Small amts. of nitrate and nitrite were also found. Expts. showed that results with iodometric method and with new indicator, which were widely different with crude sewage, agreed better when sewage had been settled and filtered or treated with coagulants; effects of suspended or colloidally dissolved org. substances was therefore studied. These substances, produced on addn. of Cl, which had no action on new

(*Continued on page 66 P&R*)

(*Continued from page 64 P&R*)

indicator but liberated I from K iodide, were found to have no bactericidal action. Expts. with settlable material from no. of sewages are described. With sewage treated with 110 mg/l Cl iodometric measurement showed presence of 17 mg/l free Cl, whereas true excess of bactericidally active Cl, as shown by new indicator, was 8 mg/l. Importance of this difference, especially in case of sewage contg. tubercle bacteria, which require 10 mg/l Cl for their destruction, is pointed out. Only bactericidally active Cl is given by new indicator.—*WPA*

The Polarographic Determination of Fluoride. I. Basic Principles of the Method: Application to the Cathode-Ray Polarograph. B. J. MacNulty, G. F. Reynolds & E. A. Terry. Analyst, **79**:190 ('54). Polarographic method has been developed for estng. fluoride, based on reduction in height of polargraphic step obtained when aluminium-Solochrome Violet R.S. complex is reduced. Linear relationship was found to exist between depression of step height and concn. of fluoride. Detailed procedures are given for estng. fluoride directly or after distiln. as hydrofluosilicic acid, in concn. range 0.1–0.8 μg/ml fluoride using std. polarograph and in ranges 0.02–0.1 μg and 0.004–0.02 μg/ml fluoride using cathode ray polarograph; pH of 3.9 was most suitable. For submicrogram amts. of fluorine, impurities must be removed from reagents by complexing with acetylacetone and extracting with chloroform. Results of anals. of std. solns. are given in tables.—*WPA*

The Polarographic Determination of Fluoride. II. The Determination of Fluoride in Bromine, Hydrochloric Acid, and Hydrobromic Acid. J. S. Beveridge et al. Analyst, **79**:267 ('54). Polarographic method of detg. fluoride has been applied to detn. of fluoride in hydrochloric acid, hydrobromic acid, and B. In case of hydrochloric acid, acid is neutralized with ammonium hydrox-

(*Continued on page 70 P&R*)


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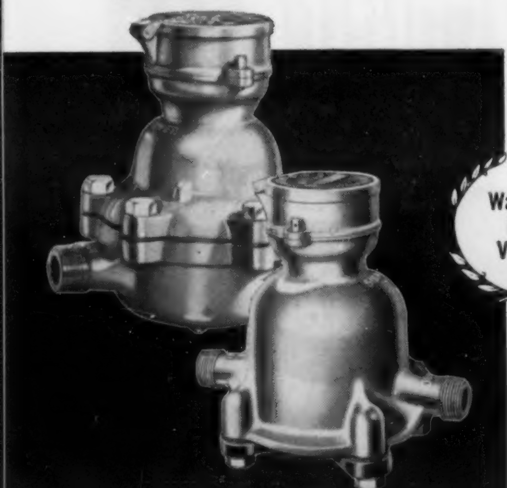

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(*Continued from page 66 P&R*)

ide. High concn. of ammonium chloride results in uniform depression of step height of aluminium complex and reproducibility is reduced from ±10% to ±15% for concns. up to 0.5 ppm fluoride. In applying same inflection point of wave was sufficiently dis-method to hydrobromic acid, it was found necessary to add small amt. of Cd so that inflection point of wave was sufficiently distinct for accurate measurement. For Br, small quant. of sample is placed under layer of water contg. Al salt and evaporated by gentle heating until no Br can be detected by starch-iodide test. Fluoride is then detd. in soln. In general, and particularly with hydrobromic acid, results with cathode ray polarograph were better than with conventional polarograph.—*WPA*

Determination of Fluoride. L. Singer & W. D. Armstrong. Anal. Chem., **26**:904 ('54). Method is described for detn. of small amts. of fluoride by conversion to hydrogen fluoride and diffusion of H fluoride. Diffusion cell is 2-oz polythene bottle. Receivers are strips of polythene sheeting with part of surface depressed and roughened to form base which is wetted with aq. soln. of Na hydroxide (2 mg). Not more than 1 ml of sample is introduced into bottle and frozen; 2 ml of 70% perchloric acid is added, and bottle is closed with screw cap and heated in oven at 50°C for 20 hr. Na fluoride is dissolved from receiver by centrifuging and shaking it with known vol. of water. Soln. is analyzed either by titration with Th nitrate using Bien's indicator or fluorometrically. Using Na fluoride solns. it was found that up to 1 mg fluoride was satisfactorily collected, avg. recovery being 101.2% of theoretical. Using larger vols. of liquid, recovery became progressively poorer due to slow rate of diffusion of H fluoride through liquid. In presence of 0.5–2 mg Na chloride, results were low unless 8–10 mg Na hydroxide was used on receiver. Presence of 10 mg glucose interfered.—*WPA*

(*Continued on page 72 P&R*)

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Determination of Iodides and Bromides in Mineral Waters. V. P. KHRAMOV. Trudy Komissii Anal. Khim., Akad. Nauk S.S.S.R., Otdel. Khim. Nauk (Russ.), **5**:262 ('54). Willard and Heyn method was modified for analysis of mineral H_2O. All-glass distn. app. was used. In 1st receiver was placed 10 ml 10% KOH, in 2nd 10 ml 3% KOH. 100-ml sample and 50–100 ml 0.4% Cl water were placed in distg. flask. To sep. Br, stopcock on inlet tube was closed and sample heated to boiling. After Br was distd., air was passed through for 30–35 min at 3–5 ml/sec while flask cooled to not less than 50°. Contents of receivers were transferred to conical flask. Then 25 ml satd. $CaCl_2$, 5 ml *N* KOCl, and 5 ml 1.5% $Zn(OAc)_2$ were added. Mixt. was dild. to 100 ml and 50% AcOH was added until most of the ppt. dissolved. Last of ppt. was just dissolved by adding 5% AcOH dropwise. Soln. was boiled 5 min, 5 ml of ½-satd. Na formate soln. (CO_2-free) was added, and soln. was boiled again for 5–10 min. Bromate and iodate (in distg. flask) were detd. iodometrically. If Fe is present, iodate titration must be done in phosphate medium. In synthetic samples, contg. 3–40 mg I and 3–60 mg Br, greatest relative error for I was 1.06%, for Br. 1.66%. Analysis of 3 H_2O samples gave precise results.—*CA*

Rapid Colorimetric Determination of Traces of Iron in Water. H. NISHIDA. Japan Analyst (Jap.), **3**:250 ('54). In method for detng. low concns. of iron in water, 200–300 ml sample is taken and 5 ml of 10% strontium nitrate and 5 ml of 10% sodium carbonate are added. After boiling the mixture for 2–3 min and allowing to stand, ppt. is sepd. by centrifuging and 2 ml of nitric acid are added. If organic matter is present, soln. of the ppt. should be boiled for a few min and diluted with water to give vol. of 5 or 10 ml, 5 or 10 ml of 10% potassium thiocyanate are added, and resulting color is measured with filter with wave-

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length of **470** mμ. Method has given accurate results for concns. of iron of 0.004–0.30 mg/l.—*WPA*

Determination of Iron in the Water for Saké Brewing by the Bipyridine Method. H. Muto & K. Sato. J. Soc. Brewing, Japan, **50**:294 ('55). Bipyridine method (Moss & Mellon, C. A., **37**:1097) was modified for visual colorimetry of slight color developed with 0.01–0.1 ppm Fe in water. Into 50 cc water in Nessler tube, 0.1% bipyridine reagent 1, 10% Na_2SO_3 soln. 5, and HCl soln. (1:3) 1 cc. were added successively, shaken, and after 10-30 min the color developed was compared visually against standards. Change of transmittance with 10-mm cell was very small (about 2%) under 0.1 ppm, and photoelectrocolorimetry did not suit in range of 0.01–0.1 ppm. Na, K, Ca, Mg, Al, NH_4, Cl, SO_4, NO_3, and PO_4 did not inhibit detn. 4 kinds of tap water were analyzed by method, and results were compared with those obtained by the thiocyanate method (Hanzal, Proc. Soc. Exptl. Biol. Med., **30**:846 ('33)).—*CA*

The Use of Trilon B for the Determination of Iron in Natural Waters. A. A. Bashkirtseva & E. M. Yakimets. Factory Lab. (U.S.S.R.), **21**:533 ('55). In detn. of low concns. of iron in natural waters with Trilon B, it has been found that ammonium thiocyanate is more suitable than other indicators that have been used. pH value of soln. should be adjusted to 1.4–1.6.—*WPA*

The Spectrophotometric Determination of Magnesium With Thiazol Yellow Dyes. T. A. Mitchell. Analyst, **79**:280 ('54). Critical examn. of spectrophotometric method of detg. Mg with thiazol yellow dyestuffs has been made. Rapid fading of Mg dye complex was shown to be associated with ageing of Mg hydroxide; it was inhibited by addn. of glycerol and concd. sodium hydroxide. Protective colloids greatly increased soly. of Mg hydroxide and, there-

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fore, of colored complex; sol. starch was preferred, because it does not affect color of dye or complex. Al, Mn, Fe, Co, Ni, Cu, and Zn in concns. of 25 μg/ml test soln. caused marked interference. Effect of calcium was variable. Small amts. of citrate and tartrate ions suppressed color formation. In proposed method, Mg is pptd. as Mg ammonium phosphate after removal of Ca by ppn. as oxalate and addn. of citrate to hold Fe and Al in soln. Ppt. is redissolved in aq. acetic acid and colorimetric detn. is carried out on resultant so'n. More than 50 μg Mn in soln. contg. 15–120 μg Mg interfered. In anals. of plant extracts, std. deviation ranged from 1.2 to 4.0 for concns. of 12–120 μg/ml Mg. Recovery of Mg added to plant extracts ranged from 93–107% and avgd. 101%.—*WPA*

Determination of Inorganic Nitrogen in Surface Waters. I. Colorimetric Determination of Nitrites and Nitrates. II. Colorimetric Determination of Ammonia. E. V. Gerovová & J. Chalupa. Csl. Hyg., Epidemiol., Mikrobiol., Imunol. (U.S.S.R.), **4**:269 ('55). Several methods of detng. nitrites and nitrates colorimetrically were reviewed. Method previously described by Sanchez was modified. Method is sensitive to 0.05 mg of nitrogen/l with error of ±1.9%. New method for colorimetric detn. of ammonia is described based on reaction of ammonium ion with phenol and hypochlorite. Sensitivity is 0.004 mg of ammonia/l, with error of ±10%.—*WPA*

Colorimetric Determination of Nitrogen as Nitrite, Nitrate, or Ammonium Salt. I. A New Method of Determining Nitrogen as Nitrite. S. Ato & H. Aoki. Rep. Sci. Res. Inst. (Jap.), **30**:329 ('54). New colorimetric method of detng. nitrite nitrogen is described. 2 reagents used are 1-naphthylamine and phenol disulphonic acid. When added to nitrous acid in this order, purple color is formed, which can be used to detn. 0.005–0.2 ppm nitrogen. When added to

(Continued on page 78 P&R)

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nitrous acid in reverse order, yellow color is formed, which can be used to detn. 0.2–1 ppm nitrogen. Certain ions, notably those of ferric and ferrous iron, aluminum, lead, and sulphur interfere if present in more than certain concns. which are given.—*WPA*

T-O_2, an Instrument for the Estimation of Temperature and Dissolved Oxygen in Natural Waters. S. B. Saila. Progr. Fish Cult., **17**:162 ('55). Illustrated description is given of design and operation of instrument, combining a simple thermistor circuit with electrolytic cell, for detn. of temp. and DO in water.—*WPA*

The Further Development of the "Oxygen Plummet" for Direct Electrochemical Determination of Dissolved Oxygen in Natural Waters. F. Todt & G. Petsch. Gesundh.-Ing. (Ger.), **76**:104 ('55). Authors describe developments in design of O plummet, objects of which were to reduce electrode area, to develop electrode which produced correct values immediately after immersion, and to cut out effect of natural movement of water and increase reproducibility by more rapid movement at electrode. Electrodes of gold amalgam with surface area of 0.1 cc. were found to give const. and uniform results. Details are given of constr. of app. in which plummet is fixed in vessel filled with water under examn. and supplied with stirrer to produce regular movement of water. Results of expts. in tap water and in Havel R. at different points are given. In river irregular movement of water caused variation and adaptation was made to plummet to ensure regular flow at rapid rate past electrodes; these were placed in center of plummet where passage was narrowed to about $\frac{1}{3}$ size of inlet and outlet openings. With increased rate of flow steady readings were obtained with reduced size of electrode, and use of gold amalgam largely removed effect of carbonate hardness. New form of plummet is suitable for direct measurements in waters of varying compn.—*WPA*

Colorimetric Determination of Low Concentrations of Dissolved Oxygen in Wa-

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(Continued from page 78 P&R)

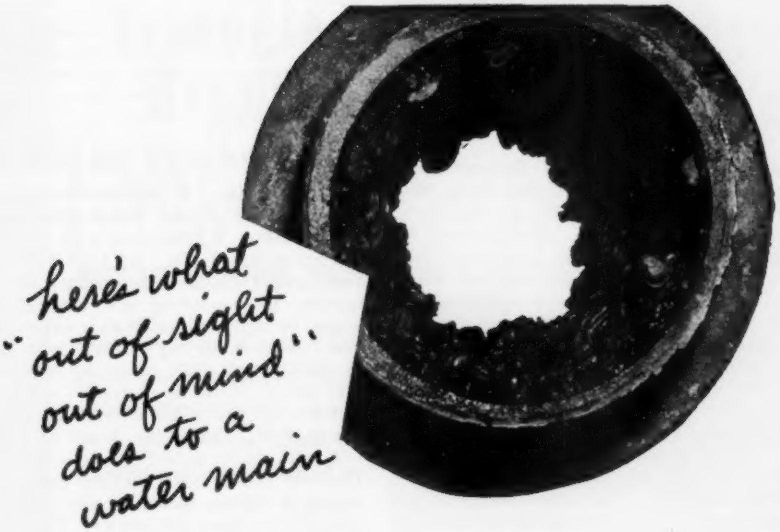
ter. L. S. Buchoff, N. M. Ingber & J. H. Brady. Anal. Chem., **27**:1401 ('55). From 0–25 parts/billion O in water is detd. in special cell by color change of soln. of indigo carmine in glycerol. Method is especially adapted for field use by nontech. personnel. —*CA*

Gasometric Determination of Dissolved Oxygen in Pure and Saline Water as a Check of Titrimetric Methods. A. B. Wheatland & L. J. Smith. J. Appl. Chem. (Br.), **5**:144 ('55). $HgCl_2$ was added to sea water to prevent loss of O by biochem. oxidation of org. matter. Dissolved gases were then extd. from known vol. of water by boiling *in vacuo*. Compn. of extd. gases was detd. in const.-vol gas-analysis app. CO_2 was absorbed by 40% KOH soln. O was absorbed with alk. soln. of $NaHSO_3$ contg. Na anthraquinonesulfonate as catalyst. The probable max. error was ±0.04 ppm O. The titrimetric method was modification of Winkler method with amperometric end point. Titrimetric method was accurate to within ±0.02 ppm O. Mean difference between results obtained by gasometric and titrimetric method was about 0.01 ppm.—*CA*

Determination of Zinc, Copper, and Lead in Some Chilean Mineral Waters. O. C. González. Bol. Soc. Chilena Quím. (Chile), **5**:22 ('53). Dithizone titration method of Granton used for detn. of zinc or lead was compared with dithizone photoelectric method Sensitivity of both methods was similar for amts. smaller than 16 μg. Other ions did not interfere appreciably with detns. Carbon tetrachloride soln. of dithizone was preferred for zinc, but chloroform solution was best for lead. Best method for detng. copper was found to be pptn. with pyridine in presence of acetic acid and ammonium cyanate, extraction with chloroform, and colorimetric estn.—*WPA*

Determination of Dissolved Solids in Water Samples by Flame Photometer. G. E. Marsh. Analyt. Chem., **27**:320 ('55). Beckman flame photometer with photomultiplier tube can be used for rapid and accurate

(Continued on page 82 P&R)



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detn. of calcium, magnesium, sodium, and iron in water. Concns. which can be detd. are, calcium 30–45 ppm, magnesium 5–15 ppm, sodium 5–25 ppm, and iron 1–2 ppm, each with accuracy of ±0.1 ppm.—*WPA*

Determination of Anionic Detergents in Sewage, Sewage Effluents and River Waters. J. LONGWELL & W. D. MANIECE. Analyst, **80**:167 ('55). Method for the detn. of anionic detergents in sewage, sewage works effluents, water, and sea water, is based on formation, and extraction by chloroform, of complex of detergent and methylene blue, but differs from other methods using this reaction in that extraction is carried out in alkaline soln. and extracts are washed with acid soln. of methylene blue. Efficiency of method was compared with that of Degens, Evans, Kommer, and Winsor, which involves extraction in acid soln. and omits washing of extracts. New method was found to be unaffected by presence of inorganic ions, and gave improved recovery of sodium alkylbenzenesulphonate in sewage and sewage works effluents. It is suggested that sodium dioctylsulphosuccinate (Manoxol O.T.), which can be obtained in high state of purity, would be suitable universal reference std. for expression of cocn. of anionic detergents.—*WPA*

LABORATORY METHODS

Mineral Water. I. Determination of Sodium Ion by Flame Spectrophotometry. M. ISHIDATE, Y. MASHIKO & Y. KANROJI. J. Pharm. Soc. Japan (Jap.), **75**:1492 ('55). Detn. of Na^+ by flame photometry by direct ignition of hot-spring and mineral-spring water, by using std. calibration curve of std. NaCl soln., gives neg. error which cannot be disregarded; only exception is common-salt spring. Good anal. results can be obtained when spring water is adjusted to pH 4 with N HCl, boiled for 10 min to drive off CO_2 and H_2S, passed through Amberlite IRA-410 (R-Cl type), thereby changing all anions in sample water to Cl^-, and finally submitted to flame photometry. This method can be applied widely to majority of neutral-salt springs in Japan, excluding acid, alum, and acid alum-vitriol springs. It is better

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than chem. detn. of Na in that it gives better accuracy and shorter time is required. Weak-acid components which cannot be substituted by above resin can be disregarded. —*CA*

Colorimetry of Aluminum in Natural Water. H. Nishida. Japan Analyst (Jap.), **4**:39 ('55). To 10-500 ml sample (more than 0.02 mg Al/l) add 2-3 ml 3% H_2O_2 to oxidize interfering ions. Then add 1-5 ml 10% $Ca(NO_3)_2$ (or $Sr(NO_3)_2$) and 10% Na_2CO_3 to adjust pH of soln. at 12-13. Heat soln. and filter off pptn. Save filtrate. If sample amt. is large, treat filtrate with 1-5 ml 10% $Ca(NO_3)_2$ and small amt. of Na_2CO_3 to coppt. Al with $CaCO_3$. Filter ppt. and dissolve it in 0.5 ml HNO_3. To above filtrate or this HNO_3 soln., add 2-3 ml 2% oxine in AcOH and adjust pH at 4.5-5 with HNO_3 or NH_4OH. Shake soln. with 10-20 ml C_6H_6 for 1 min, then measure transmittance of C_6H_6 layer with 430-mμ filter. Method can be applied to Al detn. in silicate rock.—*CA*

Estimation of Coliform Density by the Membrane-Filter and the Fermentation-Tube Methods. H. A. Thomas Jr. & R. L. Woodward. Am. J. Public Health, **45**:1431 ('55). Results of statistical anals. of 3 extensive investigations on wide variety of natural waters comparing MPN and MF techniques indicate that, on avg., former gave higher indications of density by a factor 1.0-1.9 with avg. of 1.3 for the specific techniques used in these investigations. However, difference is not regarded as important from practical viewpoint because of inherent lack of precision of individual MPN value. Moreover, disparity between MF and MPN values for most water samples tested was not significantly larger than discrepancies between results obtained with permissible variations of std. diln. method. Considerable part of disparity between MF and MPN values may be attributed to fact that, mathematically considered, MPN tends to overestimate true density; on avg. MPN values are greater than true density. With nearly all samples listed precision attained with single filter was found to be 2-5 times greater than that of 5-5-5-tube MPN. Reproducibility of MF test as measured by coeff. of variation of replicate tests was found to vary inversely with sq. root of no. of colonies counted.—*CA*

Delayed Incubation Membrane Filter Test for Coliform Bacteria in Water. E. E. Geldreich et al. Am. J. Pub. Health, **45**: 1462 ('55). A delayed-incubation coliform test on membrane filters is described. This procedure is studied in parallel with detns. of MPN's of bacteria present in stored liquid samples, and results from both were compd. with results from an initial 5-tube 3-diln. MPN test. Storage of liquid samples was at 5°, room temp. (13-32°), and at 35° for periods of 24, 48, and 72 hr. MPN coliform counts made on liquid samples held at 5° for periods up to 72 hr were quite variable, but more closely approximated initial MPN results than did counts from samples stored at 13°-32° or at 35°. Estns. of coliform detns. in water by use of delayed incubation membrane filter (MF) technique at 13°-32° indicate good agreement with the MPN on stored samples at 5° but tend to be less than initial MPN. MF storage results at room temp. and at 35° are superior to results of MPN procedures on liquid samples stored more than 24 hr at room temp. or 35°. Delayed-incubation coliform test is suggested for field application where shipment of refrigerated samples cannot be made or during periods of natural or wartime emergency.—*CA*

Report on the Development and Application of the Fluid Polariscope. K. E. Robinson. Dept. of Civil and Municipal Eng., Univ. College, London (Eng.), 1954. 257 p. Some colloidal suspensions exhibit streaming double refraction at low veloc. gradients, and it has been suggested that they might be of use in investigating problems of laminar flow. In field of flow, birefringence is caused by orientation of large number of very small, anisometric, and optically anisotropic particles and, for any given suspension, its magnitude depends on deg. of orientation induced in conditions of flow. This is function of veloc. gradient, and there is therefore direct relation between double refraction and existing hydrodynamic conditions. To establish this relation, it is necessary to know true distr. of veloc. and veloc. gradient, and

(Continued on page 88 P&R)

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(Continued from page 84 P&R)

method and app. have been developed for detg. these, under any given conditions, by progressively modifying the Newtonian distrs., on the basis of double refraction measurements, until the known boundary conditions are satisfied. Suitability of various colloidal suspensions was investigated, but many were unsuitable due to poor stability, intense ageing effects, and difficulty of prepn. Most satisfactory material was found to be California White Hector Bentonite. Example is given of use of calibration results obtained for bentonite for quan. 2-dimensional anal. of flow conditions established by presence of simple models in channel. Results of investigation are given in tables and graphs, and bibliography of 56 references is appended.—*WPA*

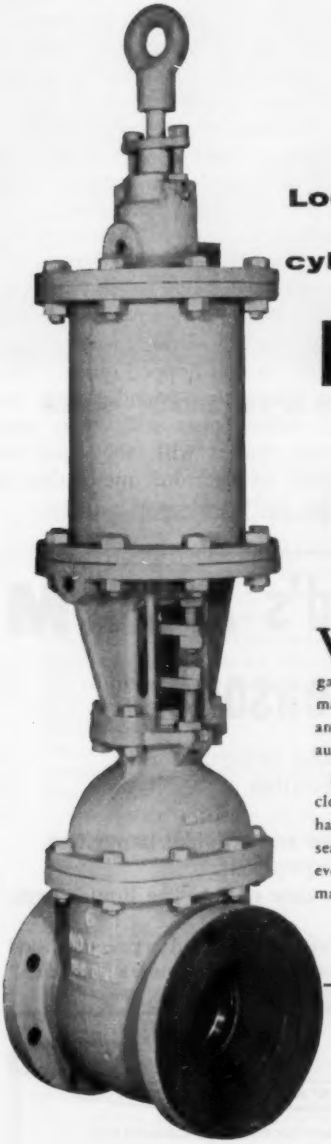
Differential Bile-Salt Test at 44° for the Detection of Fecal Contamination of Water. Z. Buczowska. Roczniki Państwowego Zakladu Hig. (Pol.), **6**:185 ('55) (English summary). Improved bact. methods for detection of fecal contamn. in water were studied. Water samples obtained from several sources were incubated in lactose broth (official method) and then differentiated by confirmatory test at 44° on Endo-agar or bile-salt broth. Compn. of broth was bile-salt 5, lactose 10, peptone 20, NaCl 5 g, distd. water 1000 ml, indicator soln. 1%. Analysis of results indicated that bile-sale test at 44° is of value for rapid detection of fecal contmn. Results were obtained within 2 days instead of usual 5 days. Total error of 2.8% was insignificant in comparison to total exptl. error.—*CA*

Oxygen Maxima in Water and the Determination of Polarographic Purity Coefficient. Z. Novák. Wasserwirtsch.-Wassertech. (Czech.), **5**:303 ('55). Possibility of utilization of polarographic methods for routine chem. anal. of waters is suggested. Technique permits polarographic detn. of purity of water and can serve as control instrument during water-purification process. Method is based on O max. in analyzed water after aeration and after shaking with active charcoal. Shaking removes contaminants which inhibit O max. Results are reported as polarographic purity coeff. (P_{KR}) corresponding to proportion between O max. in mm in water after aeration and after treatment with 0.5 g of chemically pure activated charcoal.—*CA*

Determination of Carbon Dioxide in Water by Conductivity Measurements. C. A. Noll & J. W. Polsky. Tappi, **39**:51 ('56). Method is described for detn. of CO_2 which is applicable to anal. of waterside deposits, treatment chems., and org. C in H_2O. CO_2 is absorbed in $Ba(OH)_2$ soln., and decrease in cond. caused by pptn. of $BaCO_3$ is measured. This decrease is proportional to CO_2 absorbed. CO_2 content of solid sample is calcd. as follows: 1 micromho sp. conductance difference = 0.0756 mg CO_2 at 25°. App. used is described with help of sketch, and details of procedure are given.—*CA*

Biochemical Oxygen Demand Test: A Note on Variable Results From the Use of Stored Standard Dilution Water. A. B. Wheatland & R. G. Smith. Analyst (Br.), **80**:899 ('55). Use of std. diln. water in detn. of BOD has been recommended by the APHA (1955), and expts. here described indicate that water may be stored for short time provided glass vessels have been cleaned with chromic acid and thoroughly rinsed with water. Fairly common practice of repeatedly topping up stock of std. diln. water, however, should be avoided as it may lead to unreproducible and perhaps misleading results. When nitrification occurs in blanks but not in dilns. of sample, erroneous low values may be obtained. When nitrification is complete in blanks at beginning of test but occurs in samples during incubation, very high BOD values will be obtained.—*CA*

Flame-Photometric Determination of Dissolved Solids in Water. G. E. Marsh. Appl. Spectroscopy, **10**:8–10 ('56). Flame photometer has been successfully applied to direct detn. of 30–45 ppm Ca, 5–16 ppm Mg, 5–20 ppm Na, and 1–2 ppm Fe in water. Because Fe and Mg detns. are affected by presence of Ca and Na, calibration stds. were prepd. to contain different amts. of 1 ion in presence of fixed amts. of other 3. Based on repetitive anals. of 3 synthetic and 3 natural samples, repeatability is about 0.1 ppm when oxyacetylene flame is used. No sample prepn. is needed and complete anal. takes 30 min.—*CA*



(Continued from page 50 P&R)

The first line of offense in public relations has always been the meter reader—and, as the best offense has been his inoffensiveness, he often has a rather bad time of it. Particularly of late, the infinite patience and tough skin of these meeters of the public seem to have been put to frequent test. And, as usual, it has been man's best friend that has been the meterman's worst enemy. Last year, the Post Office Department reports, 5,880 letter carriers were bitten by dogs in delivering mail to the 17,000,000 dog-owning families in the US. Imagine, then, how many meter readers—who usually must penetrate to the basement of the homes on their routes—must have cushioned a canine's canines. And, whereas mailmen have decided to make bad-dog owners come to the post office to pick up their mail, the counterpart would hardly be practical for the water utility. Little wonder, then, that meter readers have been willing to consider almost anything—short of violence, of course—to protect themselves: chemical repellants on their clothing, padded pants and shin guards, memorizing the names and habits of the dogs on their routes, using BIG escort dogs, and now, finally, offering dog candy to pacify their tormentors (*see* May P&R p. 98). Of course, none of these methods has worked—even the dog candy technique having flopped quickly when it was found that friendly dogs love it and vicious ones still prefer meat. Realizing that it will take some time to perfect an electronic meter that can

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be read from the home office and that installing meters at the curb, even for only the bad-dog customers, will be costly, we can only suggest experimentation now with a portable hydrant. A balsa wood model would be light, reasonably inexpensive, and, as demonstrated in the voyage of the *Kon-Tiki,* relatively resistant to waterlogging.

As if dogs themselves weren't enough trouble, though, their coteries now seem to be getting into the act. Thus, an Indianapolis Water Co. meter reader got in trouble with his part of the public by acting as a flea carrier, picking up his biter biters at one house and parceling them out along the way. The description that appeared in the company's house organ convinced us that Indiana fleas must be considerably larger than the varieties that we've been privileged to watch perform, but that apparently made it possible to round them up quickly enough to limit the customers' wrath.

What with other pets that peck and scratch, what with garrulous and grumpy housewives, what with wet paint and newly washed floors, what with meter hiders and cobwebby, sooty corners, it is a dog's life a meter reader leads and his worst offense rates a good defense.

Rudolph A. Schatzel, of Rome Cable Corp., Rome, N.Y., has been installed as president of ASTM. The new vice-president is Kenneth B. Woods, of Purdue University, Lafayette, Ind.

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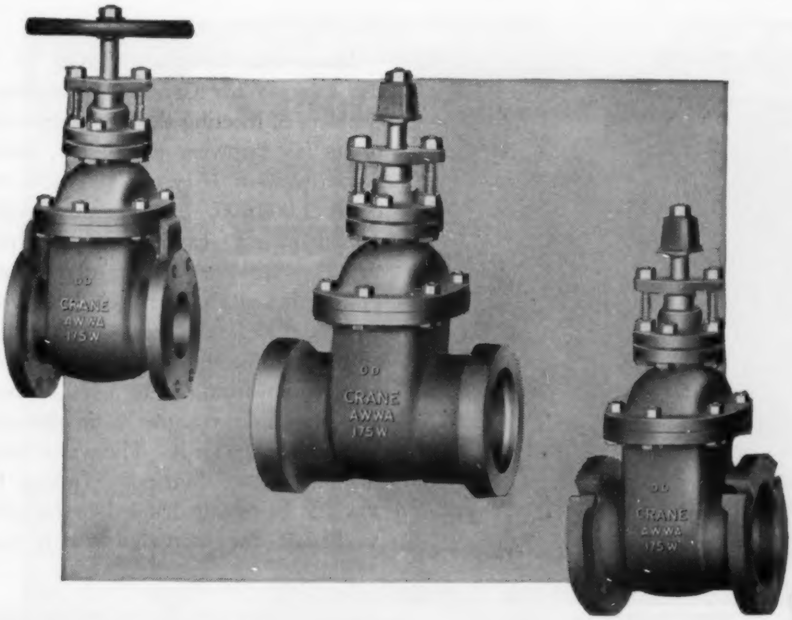
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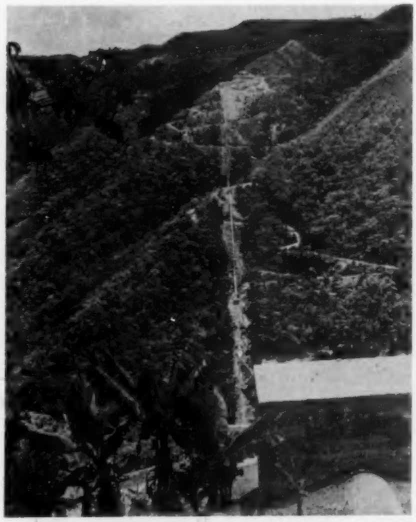

Hamilton Wright photo

Mountain climbing proved to be the solution for Caracas, Venezuela's problem of meeting the water demands of its fast growing population, which has tripled in 15 years and now exceeds 1,000,000. A 17-mile pipeline (photo at left) carries a supply from the Tuy River over Guayabo Mountain to the city's Mariposa Reservoir. The pipeline, which crosses 128 bridges and passes through two tunnels, varies in elevation from 433 to 3,470 ft above sea level and requires four pumping stations to service it. The water treatment plant at Mariposa (photo below) is being quadrupled in capacity, to handle the increased supply now available.

Hamilton Wright photo

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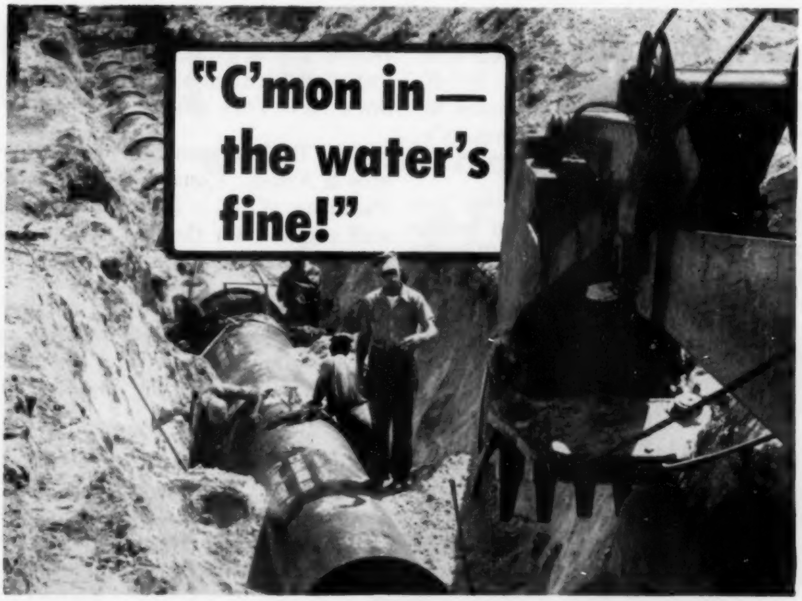

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(*Continued from page 94 P&R*)

John R. Hartley, vice-president and manager of product sales for B-I-F Industries, Inc., has been promoted to general manager of the company's Builders-Providence Div.

Harry N. Lowe Jr., chief of the San. Eng. Branch, US Army Corps of Engineers' Research & Development Labs., Fort Belvoir, Va., has been awarded the Wheeler Medal by the Society of American Military Engineers for his work on an improved suspended-solids contact clarifier.

Ludlow Valve Mfg. Co. announces the appointment of Forbes Meston as comptroller and David D. Ackerman as director of personnel. Mr. Meston was previously with Ruberoid Co., Bound Brook, N.J., while Mr. Ackerman had been sales manager of Ludlow's New York office.

Carl B. Johnston has joined Pomeroy & Assocs., Cons. & Chem. Engrs., Pasadena, Calif. He was formerly staff engineer for the Los Angeles Regional Water Pollution Control Board.

Richard A. Ball has been advanced from production engineer to sales engineer by Smith-Blair, Inc. His headquarters will be at the firm's South Gate, Calif., branch.

Russell L. Sylvester, formerly with New York Air Brake Co., has joined Rockwell Mfg. Co. as chief engineer of its Central Valve Research & Development Dept. in Pittsburgh.

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Citations for outstanding operation have been awarded to six of Florida's more than 300 water plants by the State Board of Health's Bureau of San. Eng., David B. Lee, director. The recipients, and the categories in which they were cited, are: John S. Whitney, superintendent, Community Utilities Corp., West Coral Way Subdiv., Miami (primary treatment, under 10,000 population); Thomas Paul, chemist and chief operator, Bradenton (filtration, 10,000–25,000); Miss Eleanor A. Flanigan, technician, West Palm Beach (filtration, over 25,000); Eldon L. Holoubek, operator, US Sugar Corp. water plant, Clewiston (filtration, under 10,000); John B. Sellers, superintendent, Vero Beach (lime-soda softening, under 10,000); and Clarence R. Henry, chief chemist, Dept. of Water & Sewers, Miami, and W. L. Black, superintendent, Hialeah Water Plant (lime-soda softening, over 25,000—joint citation).

S. Morgan Smith Co., York, Pa., announces the creation of a Valve Div. to handle all its valve manufacturing and marketing operations. Carl J. Wilcox, manager of valve sales, heads the new division.

Florida citer and citees: Front row, left to right—Henry, Miami; Miss Flanigan, West Palm Beach; Director Lee. Rear row—Sellers, Vero Beach; Holoubek, Clewiston; Whitney, West Coral Way; Paul, Bradenton.

Service Lines

Softeners, supplied as package units with prefabricated piping, are described in a 4-page bulletin, No. MSP-123, available from Hagan Corp., 323—4th Ave., Pittsburgh 30, Pa.

Steel pipe—its history, manufacture, installation, protection, and advantages—is the subject of a 32-page illustrated brochure, "Wherever Water Flows Steel Pipes It Best," issued by Steel Plate Fabricators Assn. The booklet, which includes a section on specifications and a bibliography of steel pipe literature, is available from the association at 79 W. Monroe St., Chicago 3, Ill.

Controlling water weeds is the subject of a revised bibliography prepared by the Research Div. of Chipman Chemical Co. The 22-page brochure, titled "List of References on Control of Aquatic Plants, Including Algae," lists more than 300 recent papers and texts on the subject of aquatic weed control. Available from the same company is a 34-page booklet, "You Can't Argue With Weeds," which includes sections on agricultural weeds, turf weeds, and woody plants. Both brochures can be obtained from the company at Bound Brook, N.J.

Safe handling and storage of chlorine is covered in an illustrated wall chart (17 × 23 in.) available from Diamond Alkali Co., 300 Union Commerce Bldg., Cleveland 14, Ohio.

'Accelator' treatment units, produced by Infilco Inc., are described in a 28-page bulletin (No. 1825), recently revised, which can be obtained from the company, at Tucson, Ariz.

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Service Lines

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General-service pumps — double-suction, single-stage—are described in a 9-page catalog, No. A-156, issued by Economy Pump Div., C. H. Wheeler Mfg. Co., 19th & Lehigh, Philadelphia 32, Pa.

Multiple V-belt drives, their origin, development, and uses, are the subject of an illustrated 36-page brochure (Bul. 20E8297) which includes eight pages of engineering tables and data, issued by Allis-Chalmers Mfg. Co. Requests should be addressed to the company at 1026 S. 70th St., Milwaukee, Wis.

Glass pipe and fittings for laboratory or pilot plant use—including metal-to-glass couplings and spigot-type high-pressure glass stopcocks—are described in a 2-page brochure, No. 80-10, issued by Fischer & Porter Co., 683 Jacksonville Rd., Hatboro, Pa.

Johns-Manville announces the publication of two new booklets: "Transite Pressure Pipe Installation Guide," a pocket-sized, 108-page illustrated manual which provides detailed instructions for every step in the handling of "Transite" asbestos-cement pipe, from receiving the pipe at the job to testing finished installations; and a 20-page catalog (No. 171A) describing the composition, physical and thermal properties, and sizes of various insulations and refractory products. Copies are available on request from Johns-Manville, 22 E. 40th St., New York 16, N.Y.

Meter testing is the subject of a 44-page manual, "Testing Water Meters," issued in a revised edition by Ford Meter Box Co. The booklet contains discussions on the selection, testing, maintenance, and repair of meters; current standard specifications; and a bibliography of articles on water meter testing and maintenance. Copies may be obtained from the company at Wabash, Ind.

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NEW MEMBERS

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Anderson, C. Harvey, Chemicals, Irving Auelke Bldg., Appleton, Wis. (Jul. '56) *P*

Aubin, Herve, City Engr., Giffard, 71 Ave. Royale, Quebec 5, Que. (Jul. '56)

Baker, Frederick W., Mgr., Bangor Water Co., 34 S. 2nd St., Bangor, Pa. (Jul. '56) *M*

Behmer, Harry L., Supt. of Utilities, Portland, Ind. (Jul. '56) *M*

Blake, Harold, Mgr., Penns Grove Water Supply Co., 76 E. Main St., Penns Grove, N.J. (Jul. '56) *M*

Blankenship, William, Sales Repr., Johns-Manville Sales Corp., Pipe Div., 402 Terminal Bldg., Charleston, W.Va. (Jul. '56) *D*

Blumberg, V. Q., Sales, Batesville White Lime Co., Batesville, Ark. (Jul. '56) *P*

Blume, Otto H. W., Water & Sewers Supt., 237 W. Commonwealth, Fullerton, Calif. (Jul. '56) *MD*

Brandt, Lowell D., Public Works Inspector, La Verne, Calif. (Jul. '56)

Brodie, Earl D., Vice-Pres., Ralph N. Brodie Co., Alvarado & W. 137th Ave., San Leandro, Calif. (Jul. '56)

Bruce, Robert M., City Engr., East Lansing, Mich. (Jul. '56) *MD*

Burry, Charles James, Engr., James F. MacLaren Assocs., 321 Bloor St. E., Toronto 5, Ont. (Jul. '56) *PD*

Burt, Burke, Box 493, Tucumcari, N.M. (Jul. '56)

Camacho, Francisco G., Graduate Student, School of Public Health & San. Eng., Univ. of Minnesota, Minneapolis, Minn. (Jr. M. Jul. '56)

Cedar Falls, City of, E. L. Cummins, Supt., 119 W. 3rd St., Cedar Falls, Iowa (Corp. M. Jul. '56) *MRD*

Cruickshank, Douglas S., City Engr., City Hall, Hayward, Calif. (Jul. '56) *RD*

Cummins, E. L.; *see* Cedar Falls (Iowa)

Cundall, William, Sales Mgr., Empire Brass Mfg. Co. Ltd., 37 Dartnell Ave., Toronto, Ont. (Jul. '56)

De Courcy, Donald M. (Mrs.), Comr. of Finance, 113 Courthouse, St. Paul, Minn. (Jul. '56) *M*

Deich, Herman, Supt., Village County Water Dist., Box 217, Crestline, Calif. (Jul. '56) *RPD*

Doby, Troy Alvin, Engr., Box 2478, Raleigh, N.C. (Jul. '56) *P*

Dodson, L. B., Supt., Campbell County Water Dist., 608 S. Fort Thomas Ave., Fort Thomas, Ky. (Jul. '56) *M*

Doxsey, Arnold S., Mech. Engr., American Water Works Service Co., Inc., 3 Penn Center Plaza, Philadelphia 2, Pa. (Jul. '56)

Dysert, William; *see* Warren (Ind.) Water Plant

Elton, Earl W., City Engr., Olympia, Wash. (Jul. '56) *M*

England, Delos H.; *see* Olin Mathieson Chemical Corp.

Fam, Michael Yue-Onn, Chief Design Engr., Hume Industries (F. E.) Ltd., Box 297, Singapore (Jul. '56) *D*

Freeman, Roy W., Mech. Engr., Ward K. Stallings Co., 3120 Maple Dr., N.E., Atlanta 5, Ga. (Jul. '56) *P*

Grady, Walter H., Sales Engr., Turbine Equipment Co., 63 Vesey St., New York 7, N.Y. (Jul. '56) *D*

Grover, Basil William, Asst. Gen. Mgr., Public Utilities Com., 272 Dundas St., London, Ont. (Jul. '56) *MRD*

Hansen, Joseph John, Supt., Beach Haven Water Works, 25 W. Bay St., Barnegat, N.J. (Jul. '56) *MPD*

Harcourt, Ben B., Foreman, Water Plant, Rifle, Colo. (Jul. '56) *MPD*

Hazen, Edwin F., Supt. of Water, 36500 Lake Shore Blvd., Eastlake, Ohio (Jul. '56) *MD*

Hendrickson, John Delward, Engr. Trainee, Mead & Hunt, Inc., 2320 University Ave., Madison, Wis. (Jr. M. Jul. '56) *MR*

Hincapie Segura, Alfredo, Ingeniero Asesor Tecnico, Departamento Munic. de Agua Potable, Guayaquil, Ecuador (Jul. '56) *P*

Hopper, George Keith, Partner, Duncan Hopper & Assocs., 1593 Wilson Ave., Downsview, Ont. (Jul. '56)

Kaplan, Bernard, Dist. Chief Public Health Engr., State Dept. of Health, 53 Lovett Ave., Little Silver, N.J. (Jul. '56) *RPD*

Karstens, August F., Asst. Supt., Water Dept., 405 S. Gilbert Ave., La Grange, Ill. (Jul. '56)

Kearns, John T., San. Engr., American Water Works Service Co., 3 Penn Center Plaza, Philadelphia 2, Pa. (Jul. '56) *RP*

Keesling, Homer Grant, Utilities Coordinator, State Office of Civil Defense, Box 110, Sacramento 1, Calif. (Jul. '56) *D*

Kerr, Samuel E., Div. Comptroller, American Water Works Service Co., Inc., 14½ N. 10th St., Richmond, Ind. (Jul. '56)

Koenig, Louis, Research, 2500 W. Woodlawn, San Antonio, Tex. (Jul. '56) *R*

Kutilek, Leonard A., Supt., Illinois Munic. Water Co., 1201 Burlington Ave., Lisle, Ill. (Jul. '56) *M*

Lindorf, Marvin Robert, Student, Univ. of Calif., Berkeley 4, Calif. (Jr. M. Jul. '56) *R*

Loeffel, Victor C., Sales Engr., Johns-Manville Sales Corp., Pipe Div., 2714 Market St., Youngstown 7, Ohio (Jul. '56) *D*

Logan, Carl, Corporation Sales, Empire Brass Co. Ltd., Dundas St. E., London, Ont. (Jul. '56)

Lott, Raymond Allen, Water Supervisor, Atlantic Refining Co., 3144 Passyunk Ave., Philadelphia, Pa. (Jul. '56) *MRPD*

Lowes, George H., Chief Engr., Water Treatment & Filtration Plant, 5140 E. River Rd., Hamilton, Ohio (Jul. '56) *MP*

Lyon, Marvin M., Supt., Water Dept., Box 611, Eureka, Kan. (Jul. '56)

MacLennan, R. S., Engr., E. M. Powell & Assocs. Ltd., 442 Oak St. E., North Bay, Ont. (Jul. '56)

Maurer, Robert Herman, Process Engr., Sec. Leader in Charge of Utilities, Celanese Corp. of America, Box 428, Bishop, Tex. (Jul. '56) *P*

McLean, Murray D., Partner, Northland Eng., 643 Cassells St., North Bay, Ont. (Jul. '56)

Moody, Robert L., Pres., Culligan Soft Water Service of Inglewood, Inc., 516 W. Manchester Blvd., Inglewood, Calif. (Jul. '56) *P*

Nightingale, Lawrence, Director of Public Works, 109 N. Main St., Hartford, Wis. (Jul. '56) *M*

Nunnenkamp, Lloyd William, Meter Supervisor, Southern California Water Co., 9610 S. Wall St., Los Angeles 3, Calif. (Jul. '56) *D*

O'Connell, Conrad William, Gen. Mgr., Onandaga County Water Authority, Box 1004, Syracuse 1, N.Y. (Jul. '56) *MR*

Olin Mathieson Chemical Corp., Delos H. England, Water Works Supt., East Alton, Ill. (Corp. M. Jul. '56) *RP*

Pace, John P., Supt. of Utilities, Light & Water Dept., Fairbury, Neb. (Jul. '56)

Paigthorp, Robert Eugene, Graduate Student, San. Eng., Oregon State College, Corvallis, Ore. (Jul. '56) *MRP*

Patino, Manuel Virgilio, Engr., Rader & Assocs., 111 N.E. 2nd Ave., Miami 32, Fla. (Jul. '56) *MRD*

Patterson, J. D.; *see* Patterson City (Calif) Water Co.

Patterson City Water Co., J. D. Patterson, Pres., Plaza Bldg., Box 246, Patterson, Calif. (Corp. M. Jul. '56) *M*

Pfeiffer, William E., Sales Engr., Calgon Inc., 107 W. 12th St., Little Rock, Ark. (Jul. '56) *RP*

Podell, Zigmund John, Water Control, Feur Treatment Plant, P. W. C. Utilities, Guam (Jul. '56) *P*

(*Continued on page 106 P&R*)

(*Continued from page 104 P&R*)

Powers, James Edward, Engr. Aide, Water Works Com., City Hall, Newport News, Va. (Jul. '56) *D*

Price, S.; *see* Prince Albert (Sask.)

Prince Albert, City of, S. Price, Operator, Water Treatment Plant, Prince Albert, Sask. (Munic. Sv. Sub. Jul. '56) *MP*

Quraishi, Gulam Moinuddin, Graduate Student, San. Eng., Univ. of Ill., Urbana, Ill. (Jr. M. Jul. '56) *MRPD*

Radmer, Norbert W., Salesman, Badger Meter Mfg. Co., Milwaukee 45, Wis. (Jul. '56) *MR*

Reichardt, Edwin W., Lubrication Engr., Gisholt Machine, 1245 E. Washington Ave., Madison, Wis. (Jul. '56) *M*

Rosecky, John E., Secy.-Treas., Carl C. Crane, Inc., 2702 Monroe St., Madison 5, Wis. (Jul. '56) *D*

Royalty, John T., Mgr., Munic. Water & Sewer Com., Shelbyville, Ky. (Jul. '56)

Russell, Robert A., Gen. Mgr., Crest Public Utility Dist., 1809 Suncrest Blvd., El Cajon, Calif. (Jul. '56) *M*

Sharp, Walter C.; *see* Stafford Water Co.

Shaw, E. W., Water Utility Mgr., Clymer Water Service Co., 833 Philadelphia St., Indiana, Pa. (Jul. '56) *M*

Shepherd, Joe S., Salesman, American Cast Iron Pipe Co., Box 1491, Dallas, Tex. (Jul. '56)

Stafford Water Co., Walter C. Sharp, Supt., Stafford Water Co., Bay Ave. & Amber Sts., Beach Haven, N.J. (Corp. M. Jul. '56) *MPD*

Struyk, C. John, Salesman, Empire Brass Mfg. Co. Ltd., Dundas St., London, Ont. (Jul. '56)

Trent, Russell John, Supt., Water Works, 108—26th St. N., Brandon, Man. (Jul. '56) *MRPD*

Vincent, Ross, Sales, Empire Brass Mfg. Co. Ltd., 37 Dartnell Ave., Toronto, Ont. (Jul. '56)

Warren (Ind.) Water Plant, William Dysert, Water Distr. Supt., Warren, Ind. (Munic. Sv. Sub. Jul. '56) *MD*

Webster, Willard W., Cons. Engr., 11½ E. Broadway, Williston, N.D. (Jul. '56) *PD*

Weeks, Glenn D., Plant Operator, Water Dept., La Grange, Ill. (Jul. '56)

Williams, W. J., Mgr., Palmyra Water Co., 13 S. Railroad St., Palmyra, Pa. (Jul. '56) *M*

Wilson, Rollo D., Water Dist. Mgr., Salem Heights Water Dist., 3725 S. Commercial St., Salem, Ore. (Jul. '56) *MD*

Woods, Philip Charles, 2nd Lt., US Army, 7812 Army Unit (Resident Engr. Sec.), APO 189, c/o Postmaster, New York, N.Y. (Jul. '56) *RP*

EMGAMY Honor Roll

Listed below are the AWWA members who have contributed to "Every-Member-Get-A-Member Year" by getting members for the Sections shown, in the period Jul. 1–31. Numbers in parentheses indicate more than one member enrolled.

California
Diemer, R. B.
Laughlin, C. A.
Philips, E. A.
Phillips, E. E.
Washington, D. R.
Weight, W. O.

Canadian
Berry, A. E. (3)
Fontaine, Leopold
Oberton, A. C. E.
Smith, F. G. (4)

Chesapeake
Schroepfer, G. J.

Florida
Drew, S. T.

Illinois
Dietz, J. C.
Krause, J. W. (2)

Indiana
Barrett, M. L.
Murdoch, J. H., Jr.

Kentucky-Tennessee
Becker, W. H.
Johnson, N. G.

Nebraska
Ott, R. C., Jr.

New Jersey
Weaver, Joseph

New York
Hopkins, E. W.

North Carolina
Young, E. G.

North Central
Flack, C. A.
Sowden, H. J.
Yegen, William

Ohio
Augenstein, H. W.
Coleman, E. L.

Pacific Northwest
Clark, L. K.
Myatt, G. R.
Westgarth, W. C.

Pennsylvania
Degan, J. M.
Flentje, M. E.
Murdoch, J. H., Jr.
Patterson, J. W.

Rocky Mountain
Kidd, Noel

Southeastern
Russell, Sherman

Southwest
Burba, F. S.
Gentry, Ted (2)
Ragen, J. L.
Stephens, Uel

Virginia
Pharr, J. M.

West Virginia
Leshkow, Nick

Wisconsin
Nordness, E. L.
Rohlich, G. A. (2)
Roth, H. S.

US Territorial
Woodall, B. W.

Foreign
Halkyard, C. C.

Index of Advertisers' Products

Activated Carbon:
Industrial Chemical Sales Div.
Permutit Co.

Activated Silica Generators:
Omega Machine Co. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

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General Filter Co.
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Permutit Co.
Walker Process Equipment, Inc.

Air Compressors:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Morse Bros. Mchy. Co.

Alum (Sulfate of Alumina):
American Cyanamid Co., Heavy Chemicals Dept.
General Chemical Div.

Ammonia, Anhydrous:
General Chemical Div.
John Wiley Jones Co.

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Wallace & Tiernan Co., Inc.

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John Wiley Jones Co.

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Walker Process Equipment, Inc.

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Harco Corp.

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Monolith Portland Midwest Co.

Cement Mortar Lining:
Centriline Corp.

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Permutit Co.
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Ross Valve Mfg. Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Chemists and Engineers:
(See Professional Services)

Chlorination Equipment:
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Fischer & Porter Co.
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Wallace & Tiernan Inc.

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Wallace & Tiernan Inc.

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A. P. Smith Mfg. Co.
Smith-Blair, Inc.
Trinity Valley Iron & Steel Co.

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James B. Clow & Sons
Dresser Mfg. Div.
Skinner, M. B., Co.

Clamps, Pipe Repair:
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Dresser Mfg. Div.
Skinner, M. B., Co.
Trinity Valley Iron & Steel Co.

Clarifiers:
American Well Works
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Cochrane Corp.
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Graver Water Conditioning Co.
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Walker Process Equipment, Inc.

Condensers:
Alco Products, Inc.
Allis-Chalmers Mfg. Co.
United States Pipe & Foundry Co.

Contractors, Water Supply:
Layne & Bowler, Inc.

Controllers, Liquid Level, Rate of Flow:
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Simplex Valve & Meter Co.

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Phelps Dodge Refining Corp.
Tennessee Corp.

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Calgon, Inc.
Philadelphia Quartz Co.

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Dresser Mfg. Div.
Philadelphia Gear Works, Inc.

Diaphragms, Pump:
Dorr-Oliver Inc.
Morse Bros. Mchy. Co.
Southern Pipe & Casing Co.

Engines, Hydraulic:
Ross Valve Mfg. Co.

Engineers and Chemists:
(See Professional Services)

Feedwater Treatment:
Allis-Chalmers Mfg. Co.
Calgon, Inc.
Cochrane Corp.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)

Ferric Sulfate:
Tennessee Corp.

Filter Materials:
Anthracite Equipment Corp.
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Filters, incl. Feedwater:
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Graver Water Conditioning Co.
Infilco Inc.
Morse Bros. Mchy. Co.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Roberts Filter Mfg. Co.
Ross Valve Mfg. Co.

Filters, Membrane (MF):
AG Chemical Co.
Millipore Filter Corp.
Carl Schleicher & Schuell Co.

Filtration Plant Equipment:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chain Belt Co.
Cochrane Corp.
Filtration Equipment Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
F. B. Leopold Co.
Omega Machine Co. (Div., B-I-F Industries)
Permutit Co.
Roberts Filter Mfg. Co.
Simplex Valve & Meter Co.
Stuart Corp.
Wallace & Tiernan Inc.

Fittings, Copper Pipe:
Dresser Mfg. Div.
M. Greenberg's Sons
Hays Mfg. Co.
Mueller Co.

Fittings, Tees, Ells, etc.:
Alco Products, Inc.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Crane Co.
Dresser Mfg. Div.
M & H Valve & Fittings Co.
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co
R. D. Wood Co.

Flocculating Equipment:
Chain Belt Co.
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Infilco Inc.
F. B. Leopold Co.
Permutit Co.
Stuart Corp.

Fluoride Chemicals:
American Agricultural Chemical Co.

Fluoride Feeders:
Fischer & Porter Co.
Omega Machine Co. (Div., B-I-F Industries)
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Co., Inc.

Furnaces:
Jos. G. Pollard Co., Inc.

Gages, Liquid Level:
Builders-Providence, Inc. (Div., B-I-F Industries)
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Gages, Loss of Head, Pressure & Vacuum, Rate of Flow, Sand Expansion:
Builders-Providence, Inc. (Div., B-I-F Industries)
Foxboro Co.
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Minneapolis-Honeywell Regulator Co.
Jos. G. Pollard Co., Inc.
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Gasholders:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Gaskets, Rubber Packing:
James B. Clow & Sons
Johns-Manville Corp.

Gates, Shear and Sluice:
Armco Drainage & Metal Products, Inc.
Chapman Valve Mfg. Co.
James B. Clow & Sons
Morse Bros. Mchy. Co.
Mueller Co.
R. D. Wood Co.

Gears, Speed Reducing:
DeLaval Steam Turbine Co.
Philadelphia Gear Works, Inc

Glass Standards—Colorimetric Analysis Equipment:
Klett Mfg. Co.
Wallace & Tiernan Inc.

Goosenecks (with or without Corporation Stops):
James B. Clow & Sons
Hays Mfg. Co.
Mueller Co.

Hydrants:
James B. Clow & Sons
Darling Valve & Mfg. Co.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
Rensselaer Valve Co.
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Hydrogen Ion Equipment:
Wallace & Tiernan Inc.

Hypochlorite; see Calcium Hypochlorite; Sodium Hypochlorite

Ion Exchange Materials:
Allis-Chalmers Mfg. Co.
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.

Iron, Pig:
Woodward Iron Co.

Iron Removal Plants:
American Well Works
Chain Belt Co.
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
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Jointing Materials:
Hydraulic Development Corp.
Johns-Manville Corp.
Keasbey & Mattison Co.
Leadite Co., Inc.

Joints, Mechanical, Pipe:
American Cast Iron Pipe Co.
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Leak Detectors:
Jos. G. Pollard Co., Inc.

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Dorr-Oliver Inc.
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Magnetic Dipping Needles:
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Neptune Meter Co.
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Worthington-Gamon Meter Co.

Meter Reading and Record Books:
Badger Meter Mfg. Co.

Meter Testers:
Badger Meter Mfg. Co.
Ford Meter Box Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.

Meters, Domestic:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Meters, Filtration Plant, Pumping Station, Transmission Line:
Builders-Providence, Inc. (Div., B-I-F Industries)
Foster Eng. Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.

Meters, Industrial, Commercial:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Builders-Providence, Inc. (Div., B-I-F Industries)
Fischer & Porter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Simplex Valve & Meter Co.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Mixing Equipment:
Chain Belt Co.
General Filter Co.
Infilco Inc.
F. B. Leopold Co.

Paints:
Barrett Div.
Inertol Co., Inc.

Pipe, Asbestos-Cement:
Johns-Manville Corp.
Keasbey & Mattison Co.

Pipe, Brass:
American Brass Co.

Pipe, Cast Iron (and Fittings):
Alabama Pipe Co.
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
United States Pipe & Foundry Co.
R. D. Wood Co.

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Pipe Jointing Materials; see Jointing Materials

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Pumps, Chemical Feed:
Infilco Inc.
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Layne & Bowler, Inc.

Pumps, Diaphragm:
Dorr-Oliver Inc.
Morse Bros. Mchy. Co.
Wallace & Tiernan Inc.

Pumps, Hydrant:
W. S. Darley & Co.
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Pumps, Hydraulic Booster:
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Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.

Pumps, Sump:
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.

Pumps, Turbine:
DeLaval Steam Turbine Co.
Layne & Bowler, Inc.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
Builders-Providence, Inc. (Div., B-I-F Industries)
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Sand Expansion Gages; see Gages

Sleeves; see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
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Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
General Filter Co.
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Permutit Co.

Sodium Aluminate:
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Wallace & Tiernan Inc.

Sodium Silicate:
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Cochrane Corp.
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Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon, Inc.
Cochrane Corp.
General Filter Co.
Infilco Inc.
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Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Bethlehem Steel Co.
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Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

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R. D. Cole Mfg. Co.
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Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

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Pittsburgh-Des Moines Steel Co.

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Mueller Co.

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Builders-Providence, Inc. (Div., B-I-F Industries)
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Industrial Chemical Sales Div.
Infilco Inc.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

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R. D. Wood Co.

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Ross Valve Mfg. Co., Inc.
S. Morgan Smith Co.

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James B. Clow & Sons
DeZurik Shower Co.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
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Rensselaer Valve Co.
S. Morgan Smith Co.
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Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.

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Henry Pratt Co.
Ross Valve Mfg. Co., Inc.

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Crane Co.
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
DeZurik Shower Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Large Diameter:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Regulating:
DeZurik Shower Co.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Minneapolis-Honeywell Regulator Co.
Mueller Co.
Henry Pratt Co.
Ross Valve Mfg. Co.
S. Morgan Smith Co.

Valves, Swing Check:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:
Builders-Providence, Inc. (Div., B-I-F Industries)
Infilco Inc.
Simplex Valve & Meter Co.

Waterproofing:
Barrett Div.
Inertol Co., Inc.

Water Softening Plants; **see Softeners**

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
Wallace & Tiernan Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Cochrane Corp.
Dorr-Oliver Inc.
Fischer & Porter Co.
General Filter Co.
Graver Water Conditioning Co.
Hammond Iron Works
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:
Layne & Bowler, Inc.

Well Screens:
Johnson, Edward E., Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite; **see Ion Exchange Materials**